**US Army Corps
of Engineers**

SEISMIC STABILITY EVALUATION OF FOLSOM DAM AND RESERVOIR PROJECT

Report 3

CONCRETE GRAVITY DAM

by

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The man-made water retaining structures at the Folsom Dam and Reservoir Project, located on the American River about 20 miles upstream of the City of Sacramento, CA, have been evaluated for their seismic safety in the event of a magnitude 6.5 earthquake occurring on the East Branch of the Bear Mountains Fault Zone. This report documents the structural seismic analysis of the concrete gravity section of the project. The seismic evaluation of a concrete gravity section is conducted to estimate the maximum principal tensile stresses in the outer faces of the dam. These principal stress estimates show where significant cracking, if any, will occur. Based upon these analyses, it is concluded that the dam will maintain its structural integrity during and after a major earthquake.

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PREFACE

The US Army Engineer Waterways Experiment Station (WES) was authorized to conduct this study by the US Army Engineer District, Sacramento (SPK), by Intra-Army Order for Reimbursable Services Nos. SPKED-F-82-2, SPKED-F-82-11, SPKED-F-82-34, SPKED-F-83-15, SPKED-F-83-17, SPKED-F-83-14, and SPKED-D-85-12. This report is one in a series of reports which documents the seismic stability evaluations of the man-made water retaining structures of the Folsom Dam and Reservoir Project, located on the American River, in California. The Reports in this series are as follows:

Report 1: Summary

Report 2: Interface Zone

Report 3: Concrete Gravity Dam

Report 4: Mormon Island Auxiliary Dam - Phase I

Report 5: Dike 5

Report 6: Right and Left Wing Dams

Report 7: Upstream Retaining Wall

Report 8: Mormon Island Auxiliary Dam - Phase II

The work on these reports is a joint endeavor between SPK and WES. Messrs. John White and John S. Nickell, of Civil Design Section 'A', Civil Design Branch, Engineering Division (SPKED-D) at SPK were the overall SPK project coordinators. The WES Research Team Leader was Dr. Mary Ellen Hynes of the Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), WES. This portion of the study was completed by Dr. Robert L. Hall and Mr. Stanley C. Woodson of the Structural Mechanics Division (SMD), Structures Laboratory (SL), WES, and Professor Jim Nau, North Carolina State University. Initial calculations were performed by Mr. Vincent P. Chiarito, SMD, SL. Computer runs were made by Mr. Tommy Bevins and Mr. Stephen Wright SMD, SL. Computer graphics were done by Mr. Tommy Bevins. Mike Sharp (EEGD) provided foundation studies. Also, Professor Jerome M. Raphael, Professor Emeritus of the University Of California, Berkeley, recommended values for mass concrete properties used in the seismic evaluation.

Professors H. Bolton Seed, Anil K. Chopra, and Bruce A. Bolt of the University of California, Berkeley; Professor Clarence R. Allen of the California Institute of Technology; and Professor Ralph B. Peck, Professor

Emeritus of the University of Illinois, Urbana, served as Technical
Specialists and provided valuable guidance during the course of the
investigation.

Overall direction at WES was provided by Dr. Arley G. Franklin, Chief,
EEGD, and Dr. William F. Marcuson III, Chief, GL. Direction within SL was
provided by Dr. Jim P. Balsara, Chief, SMD, and Mr. Bryant Mather, Chief, SL.

COL Dwayne G. Lee, EN, is Commander and Director. Dr. Robert W. Whalin
is Technical Director.

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SEISMIC STABILITY EVALUATION OF FOLSOM DAM AND RESERVOIR PROJECT

Report 3: Concrete Gravity Dam

PART 1: INTRODUCTION

Objective

1. This report is one of a series of reports that document the investigations and results of a seismic stability evaluation of the man-made water retaining structures at the Folsom Dam and Reservoir Project, located on the American River in Sacramento, Placer and El Dorado Counties, California, about 20 airline miles northeast of the City of Sacramento. This seismic safety evaluation was performed as a cooperative effort between the US Army Engineer Waterways Experiment Station (WES) and the US Army Engineer District, Sacramento (SPK). Professors H. Bolton Seed, Anil K. Chopra, and Bruce A. Bolt of the University of California, Berkeley, Professor Clarence R. Allen of the California Institute of Technology, and Professor Ralph B. Peck, Professor Emeritus of the University of Illinois, Urbana, served as Technical Specialists for the study.

2. The purpose of the study documented in this report is to evaluate the dynamic response of Folsom Dam in the event of a Maximum Credible Earthquake (MCE) to assess the structural stability of the main concrete portion of the dam and to determine if remedial measures are necessary to prevent catastrophic loss of the reservoir. In accordance with ETL 1110-2-303 (US Army Corps of Engineers 1985) the seismic evaluation of a concrete gravity section is conducted to estimate the maximum principal tensile stresses in the outer faces of the dam. These principal stress estimates show where significant cracking, if any, will occur. Also, a sliding stability analysis is conducted in accordance with ETL 1110-2-256 (US Army Corps of Engineers 1981).

Scope

3. A comprehensive description of the concrete structure is presented. Field and laboratory investigations conducted to determine

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material properties not contained in the construction records are
A seismic analysis, based on an analytical approach utilizing a state-of-the-
art, two-dimensional finite element program, EGAD-84 (Fenves and Chopra 1984),
of the critical monolith of the Concrete Gravity Dam is presented. The time-
history response of the dam subjected to the specified earthquake ground
motions is determined, including the simultaneous effects of dam-reservoir
interaction, dam-foundation rock interaction, and reservoir bottom absorption.
The finite element solutions provide a reliable estimate of the maximum
principal stresses which occur on the upstream and downstream faces of the dam.
This study is limited to the analyses of the internal stresses in the monolith
and the overturning and sliding of the monolith on horizontal planes within the
monolith or at the monolith-foundation contact, but not beneath.

Background

Project history and location

4. Folsom Reservoir is a multipurpose reservoir located in the
Sacramento-San Joaquin Basin, California. The Folsom Project was designed and
built by the Corps of Engineers during the period from 1948 to 1956 under the
authority of the Flood Control Act of 1944 and the American River Basin
Development Act of 1949. In May 1956, ownership was transferred to the US
Bureau of Reclamation for operation and maintenance. The reservoir operates
to control flood flows in the lower American River and to provide irrigation
and hydroelectric power generation. Releases from the reservoir also help to
maintain fish-runs in the American River below the dam and to maintain
navigation along the lower reaches of the Sacramento River. A vicinity map of
the Folsom Project is shown in Figure 1. The concrete gravity section of the
Folsom Dam is located on the American River about 20 miles upstream from the
city of Sacramento, California, and serves a drainage area of approximately
1,875 square miles.

Hydrology

5. Hydrologic records obtained during the 29-year period from 1956 to
1984 show that the pool typically reaches the 466-ft elevation about 10
percent of the time during the month of June and considerably less than 10
percent of the time during the other months of the year. Based on these

records, 466 ft was considered to be a practical worst case reservoir elevation for the seismic evaluation.

Structural design history and criteria

6. The procedure used to design the structure is discussed in US Army Engineer District, Sacramento (1950). The shape of the overflow and nonoverflow sections were basically determined in accordance with the Office, Chief of Engineer's Bulletin, Civil Works No. 48-1, dated 3 May 1948, and the Engineering Manual for Civil Works. The uplift assumptions used to compute the stability of the concrete sections were modified from the criteria in the Engineering Manual due to the expected high tailwater conditions. Uplift was assumed equal to the full tailwater uplift over 100 percent of the base plus two-thirds of the differential hydrostatic head applied at the upstream toe of the dam and varying uniformly to zero at the downstream toe. The criterion used to establish the cross section of the nonoverflow section was that the resultant fall within the central 33-1/3 percent of the base and the maximum sliding factor* not be greater than 0.75 when the reservoir level is at spillway design flood pool elevation 475.4 ft and the maximum tailwater is at elevation 283.5 ft. The controlling criterion used for establishing the section of the overflow concrete section was that the resultant fall within the central 50 percent of the base and the sliding ratio not be greater than 0.85 under the following conditions: Reservoir at gross pool elevation 466.0 ft, tailwater elevation at 211.0 ft, and earthquake pseudo-static acceleration of 0.05 g acting upstream. A unit weight of 158 pcf was used for the unit weight of concrete in all computations.

Geology

7. Kiersch and Treasher (1955) studied the site geology of the Folsom Project at the time of construction. Also, construction records serve as a source for details of the site geology. A detailed foundation report, explaining the as-constructed foundation conditions, is provided in Appendix A. The project is located in the low, western-most foothills of the Sierra Nevada in central California. A late Pliocene-Pleistocene course of the American River flowed through the Blue Ravine and joined the present American River

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* Maximum sliding factor equals ratio of the forces causing sliding to the forces resisting sliding.

channel downstream of the City of Folsom. The Blue Ravine was filled with late Pliocene-Pleistocene gravel. With downcutting and headward erosion, Blue Ravine was eventually isolated and drainage was diverted to the present American River channel.

8. The Concrete Gravity Dam, the Wing Dam, and dikes 1 through 7 are founded on weathered granitic rock consisting of quartz diorite. Dike 8 and the Mormon Island Auxiliary Dam are underlain by metamorphic rock of the Copper Hill Volcanics, formerly included within the Amador Group. Appendix A gives a complete discussion of the geology beneath each concrete monolith.

Seismology

9. Tierra Engineering Consultants, Inc. (1983) performed a seismological study and determined that the maximum credible earthquake for the project is an earthquake of Local Magnitude 6.5 at a distance of about 15 km on the East Branch of the Bear Mountains fault zone. A 12-mile wide by 35-mile long area, shown in Figure 2, centered on the Folsom Reservoir was investigated using techniques including areal imagery analysis, ground reconnaissance, geologic mapping, and detailed fault capability assessment. Figure 3 shows a close-up of the study area surrounding the Folsom Project. Figure 4 shows the regional geology of the study area. Also, studies by others relevant to the seismicity of the region were reviewed in the Tierra Report. The East Branch of the Bear Mountains fault zone is the closest known capable fault and has been determined to be capable of generating a maximum magnitude M_L = 6.5 earthquake. The return period for this maximum earthquake is estimated to exceed 400 years (Tierra Engineering Consultants, Inc. 1983).

10. The seismicity studies also indicated that reservoir-induced earthquakes generated by Folsom Lake are unlikely, since the faults that underlie the project were determined to be noncapable. The minimum distance between the East Branch of the Bear Mountains fault zone and the Concrete Gravity Dam is approximately 9.5 miles. The focal depth of the earthquake is estimated to be about 6 miles. This hypothetical maximum magnitude earthquake would cause more severe shaking at the project than earthquakes originating from other known potential sources.

Selection of design ground motions

11. The seismological and geological investigations summarized in the Tierra report were provided to Professors Bruce A. Bolt and H. B. Seed (1983) to determine appropriate ground motions for the seismic safety evaluation of

the Folsom Dam Project. The East Branch of the Bear Mountains fault zone is in an extensional tectonic setting, and has a seismic source mechanism that is normal dip-slip. The slip rate from historic geomorphic and geological evidence is very small, less than 10^{-3} cm/year, with the most recent known displacement occuring between 10,000 and 500,000 years ago in the late Pleistocene period.

12. Based on their studies of the horizontal ground acceleration recorded on an array of accelerometers normal to the Imperial Valley fault during the Imperial Valley earthquake of 1979, as well as recent studies of a large body of additional ground motion recordings, Bolt and Seed (1983) recommend the following ground motions:

> Peak horizontal ground acceleration = 0.35 g.
>
> Peak horizontal ground velocity = 20 cm/sec.
>
> Bracketed duration (0.05 g) = 16 seconds.

Because of the presence of granitic plutons at the site, it is expected that the earthquake accelerations might be relatively rich in high frequencies. Bolt and Seed (1983) provided two accelerograms that are representative of the design ground motions expected at the site as a result of a maximum magnitude M_L equal to 6.5 occuring on the East Branch of the Bear Mountains fault zone. The accelerograms are designated as follows:

> M6.5 - 15K - 83A. This accelerogram is representative of the 84 percentile level of ground motions that could be expected to occur at a rock outcrop as a result of a magnitude 6.5 earthquake occuring 15 km from the site. It has the following characteristics:
>
> Peak acceleration = 0.35 g.
>
> Peak velocity = 25 cm/sec.
>
> Duration = 16 sec.
>
> M6.5 - 15K - 83B. This accelerogram is representative of the 84 percentile level of ground motions that could be expected to occur at a rock outcrop as a result of magnitude 6.5 earthquake occuring 15 km from the site. It has the following characteristics:
>
> Peak acceleration = 0.35 g.

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Peak velocity = 19.5 cm/sec.

Duration = 15 sec.

Figure 5 shows plots of acceleration as a function of time for the two design accelerograms.

PART II: GRAVITY DAM DESCRIPTION

Concrete Gravity Dam

13. The Concrete Gravity Dam lies between Stations 285 + 35.00 and
299 + 35.00. It consists of twenty-eight 50-ft-wide monoliths numbered
consecutively from the right abutment. Plan, elevation, and section views are
shown in Figures 6 through 9, respectively. The monoliths were constructed in
5-ft lifts and are founded on hard granodiorite rock. Specifications allowed
not more than 9 days to elapse between the placement of each successive
lift. The concrete was discharged from the mixing plant directly into the
hopper or bucket that conveyed it to its final point of deposition. A 2- to
10-ft-thick shell of concrete with a high cement content (rich) was placed
along the upstream and downstream faces of the monoliths from the base to the
crest (Woodward-Clyde Consultants 1983). A lean (low cement content) concrete
was placed throughout the rest of the dam section. The concrete was composed
of portland cement, water, fine and coarse aggregate, and an air-entraining
admixture. The designs of the concrete mixtures were based on the water-
cement ratio necessary to secure a plastic, workable mixture suitable for the
specific conditions of placement and to produce a concrete having durability,
impermeability, and strength.

14. The maximum height of the concrete section is 340 ft with a gross
crest length of 1,400 ft and a crest width of about 32 ft. The crest
elevation of the nonoverflow section is 480.5 ft. The overflow section has a
crest elevation of 418.0 ft with a length of 392 ft. Spillway releases are
controlled with eight tainter gates (5 to 42 ft by 50 ft; 3 to 42 ft by 53 ft)
with a flow capacity of 567,000 cfs.

Wing Dams

15. The Concrete Gravity Dam is bounded by the Right Wing Dam from
Station 218 + 00.00 to 285 + 35.00 and the Left Wing Dam from Station
299 + 35.00 to 320 + 23.29. Monoliths 1 through 6 are embedded with the Right
Wing Dam and monoliths 22 through 28 are embedded with the Left Wing Dam. The
wing dams are zoned embankments founded on weathered quartz diorite granite.
The Right Wing Dam is approximately 6,700 ft long and approximately 195 ft

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high. The core consists of well-compacted decomposed granite and fine-grained
materials from the American River channel. The Left Wing Dam is approximately
2,100 ft long, approximately 167 ft high, and is composed of well-compacted
decomposed granite.

PART III: FIELD AND LABORATORY INVESTIGATION

Background

16. The Geotechnical Laboratory (GL), WES, summarized information from construction records, engineering geology, subsurface investigations, excavation, and final preparation of the foundation rock underlying the concrete section of the Folsom Dam (Appendix A). Appendix A discusses the degrees of rock weathering and geologic features found below each monolith and the orientation of joints, faults, and shear zones below each monolith.

17. Based on a review of construction records, a field and laboratory investigation program was necessary. The objective of the investigation was to determine whether accurate material properties of the concrete and foundation were being used in the seismic analysis procedures. Tests were conducted on the concrete to determine the following material properties under rapid loading conditions: Modulus of elasticity, Poisson's ratio, compressive strength, and splitting tensile strength. Further tests were also conducted to determine the foundation rock's modulus of elasticity. The first field inves'igation was conducted in 1983 by P. C. Exploration, Inc., with Woodward-Clyde Consultants providing technical supervision of the core drilling work and conducting borehole jacking tests and data analyses. Under this contract (No. DACW05-83-R-0015), a total of twenty-eight 6-in.-diameter and six NX-size core holes were drilled at locations along the crest in the interior and on the downstream face of the dam. The location of the core holes are shown in the plan and section views of Figures 10 and 11. Woodward-Clyde then conducted in situ testing of the foundation and abutment bedrock in the six NX core holes using a Goodman jack. Woodward-Clyde prepared a written report (Woodward-Clyde Consultants 1983) summarizing the results of the field investigation. Appendix B is a summary of the Woodward-Clyde Report. From these borings, 87 boxes of concrete cores were sent to the US Army Engineer Division, South Pacific (SPD) for determination of splitting tensile strengths (Appendix C). Also, 33 cores were tested by the US Bureau of Reclamation (USBR) to determine material properties of the concrete (Peabody and Travers 1984). In 1986, 12-in.-diameter cores were drilled into the dam at locations shown in Appendix D. These cores were tested at the Structural Engineering Laboratory, University of California, Berkeley (UC Berkeley). The US Army

Engineer District, Sacramento, under contract DACW05-86-P-1049, directed
Professor Jerome Raphael (of UC Berkeley) to supervise the testing of these
cores and recommend concrete material properties for use in a seismic analysis
of Folsom Dam based on these, and earlier, test results. Professor Raphael's
report is included as Appendix D (Raphael 1986). Finally, in May 1987, the GL
was asked by the Structures Laboratory (SL) to provide the modulus of
elasticity values and Poisson's ratio for the foundation rock (Appendix E).

Summary of SPD, USBR, and UC Berkeley Tests

18. As mentioned earlier, 87 boxes of concrete cores were sent to SPD
in 1983. The cores were cut into 12-in.-length samples. SPD forwarded 77
samples to USBR and retained 36 samples, 21 of which were suitable for testing
(Appendix C). All 21 samples were cut from "lean-mix" concrete cores. Eleven
of the 21 samples were tested to determine static modulus of elasticity values
and Poisson's ratio values. The remaining 10 samples were used in the
determination of the splitting tensile strength of the concrete. The average
test values for the modulus, Poisson's ratio, and the splitting tensile
strength were 4.18×10^6 psi, 0.17, and 483 psi, respectively.

19. Raphael (Appendix D) summarizes the results of the USBR and the UC
Berkeley tests. The discussion includes results of laboratory tests on
concrete cores taken from Englebright, Folsom, and Pine Flat Dams. The values
for the elastic modulus (static and dynamic) from the USBR tests are
considerably smaller than the UC Berkeley test values for samples taken from
the lean-mix concrete. The values for Poisson's ratio determined by the two
laboratories are similar. The UC Berkeley data show a dynamic strength gain
of approximately 48 percent for the tensile strength; whereas, the USBR data
show a dynamic strength gain of approximately 6 percent. UC Berkeley tested
12-in.-diameter samples, and USBR tested 6-in.-diameter samples. Also, the
gage length of the USBR foil gages used in the tests was 4 in., which is less
than the 6-in. maximum size aggregate of the samples. UC Berkeley used a
compressometer with a 12-in. gage length. Raphael suggests that the
measurements made with gages smaller than the maximum size aggregate are
influenced by the deformation of a single large particle rather than being
responsive to the entire mass. The more likely explanation for the
differences between the two groups of tests is that the USBR specimens were

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dry, but the UC Berkeley specimens were kept saturated until tested. It is generally known that significantly higher strength values will result from static tests on dry samples as compared with static tests on wet specimens. The moisture content of the specimens has little effect on the strength as determined by dynamic tests. Table 1 compares the material property values obtained from the three test facilities for the lean mix concrete cores from Folsom Dam.

20. Comparative values for the rich-mix concete from each laboratory are not available, except for the splitting tensile strength. The USBR tests resulted in a dynamic elastic modulus of approximately 6.01×10^6 psi and a dynamic Poisson's ratio of approximately 0.22 for the rich-mix concrete. UC Berkeley determined that the static splitting tensile strength was approximately 452 psi. The dynamic splitting tensile strength values for the two laboratories were similar, being approximately 655 and 649 psi for USBR and UC Berkeley, respectively.

21. Raphael (Appendix D) explains the need for an apparent tensile strength value computed from the test strengths for comparison with concrete stresses predicted by linear analyses. Table 2 presents the concrete material properties recommended in Appendix D, including the apparent tensile strength values.

Location of Rich Concrete

22. A 2- to 10-ft-thick shell of rich concrete was placed along the upstream and downstream faces of the monoliths as mentioned earlier. The field logs of the concrete core borings presented by Woodward-Clyde Consultants (1983) indicate that the amount of rich concrete varies significantly among monoliths and within each monolith. Table 3 summarizes data from the field logs, describing the location of rich concrete. Data are not available for many of the monoliths. Because of the method of placement for the monolith sections, an exact thickness for the rich concrete layer cannot be determined; therefore, the properties of lean concrete were used in this study.

Foundation Rock Properties

23. The GL prepared estimates for several properties of the rock beneath the Folsom Concrete Gravity Dam (Appendix E). The values presented in Table 4 were computed or estimated based on field investigations, experience, and well-known expressions. A dynamic elastic modulus of 7.9×10^6 psi was recommended with lower and upper bound values of 5.8×10^6 and 11.0×10^6 psi.

PART IV: STRUCTURAL EVALUATION

Description of Analysis

24. The seismic analyses of the critical nonoverflow monolith of the dam were conducted using a state-of-the-art, two-dimensional finite element program, EAGD-84 (Fenves and Chopra 1984). In this approach, the time history response of the dam subjected to the specified earthquake ground motions is determined with the simultaneous effects of dam-water interaction, dam-foundation rock interaction, and reservoir bottom absorption included. Water compressibility is included in the analysis since the earthquake response of concrete dams can be significantly affected by this factor. The foundation rock supporting the dam is idealized as a homogeneous, isotropic, visco-elastic half plane. The dam monolith is idealized with an assemblage of four-node nonconforming planar finite elements. Dissipation of strain energy in the concrete is modeled with a constant hysteretic damping factor, S. A viscous damping ratio for all the natural vibration modes of the concrete dam on a rigid foundation with no reservoir corresponds to a constant hysteretic damping factor of twice the viscous damping ratio (Fenves and Chopra 1984). Defined at the nodal points of the dam base, the frequency-dependent stiffness matrix for the foundation work, idealized as a visco-elastic half-plane, appears in the equations of motions for the dam (Fenves and Chopra 1984). The total dam-foundation system is idealized as shown in Figure 12. As illustrated in Figure 12, the ground motions are input at the base of the dam. These two-dimensional finite element solutions provide a reliable estimate of the maximum principal stresses which occur on the upstream and downstream faces of the dam.

System Properties

25. Appendix F describes the analysis performed to verify that the tallest monolith (monolith 11) is the critical cross section. The critical dam monolith (monolith 11) was idealized as an assemblage of 240 planar, four node, finite elements. A recent study (Fenves and Chopra 1986) shows that a mesh of this fineness is adequate to capture the predominant modes of vibration and to accurately evaluate the stresses throughout the monolith.

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The linear-elastic properties of the dam concrete were obtained from static and rapid load tests on 12-in.-diameter core samples and are summarized in Table 1. The parameters shown in Table 2 represent recommended values for analysis. Since the majority of the critical tensile stresses in the dam results from the dynamic effects of earthquake loading, the modulus of elasticity used in the analysis corresponds to the value obtained from rapid load tests. Also shown in Table 2 are the recommended tensile strengths of the concrete. Table 3 indicates that the amount of rich concrete varies throughout the dam. Based on the available information, the allowable maximum principal stress in the concrete must be based on the lean concrete tensile strength. From information provided by the US Army Engineer District, Sacramento, the unit weight of the dam concrete is 158 pcf.

26. The foundation rock properties are shown in Table 4 and represent the range of the expected values. The seismic analyses are conducted using these three sets of foundation properties to assess the sensitivity of the results to the foundation stiffness.

27. Energy dissipation in the dam and foundation materials is represented by constant hysteretic damping. Constant hysteretic damping factors $S = 0.1$ for the dam concrete and $F = 0.1$ for the foundation rock are assumed (Fenves and Chopra 1984). These hysteretic damping factors correspond to a 5 percent viscous damping ratio in all natural modes of vibration and are appropriate values for the relatively large motions and high stresses experienced by the dam during strong earthquake ground motion.

28. The absorptive nature of the reservoir bottom is characterized by the wave reflection coefficient α. The coefficient represents the dissipation of hydrodynamic pressure waves in the reservoir bottom and is modeled approximately by a boundary condition of the reservoir bottom which partially absorbs incident hydrodynamic pressure waves (Fenves and Chopra 1984). The wave reflection coefficient is defined as the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a vertically propagating pressure wave incident on the reservoir bottom. It is difficult to determine reliable values of α since the bottom materials are generally comprised of variable layers of exposed rock, alluvium, and other sediments.

The material at the bottom of the reservoir determines the wave coefficient α by the following equation (Fenves and Chopra 1986):

$$\alpha = \frac{1-K}{1+K}$$

Where

$K = \rho c / \rho_r c_r$

c = Velocity of pressure waves in water (4,720 ft/sec)

ρ = Density of water (62.4 lb/ft^3)

$c_r = \sqrt{E_r / \rho_r}$

E_r = Young's modulus of reservoir bottom material

ρ_r = Density of the reservoir bottom material

29. For foundation rock modulus, E_{rock} values of 5.8, 7.9, and 11 million psi, the above equation leads to α values of 0.75, and 0.79 and 0.82, respectively. These values account for wave absorption in the rock at the reservoir bottom but do not account for additional absorption due to sedimentation.

Earthquake Ground Motions

30. Two horizontal accelerograms, designated herein as EQ-1 and EQ-2, are representative of earthquakes that might occur near the site of Folsom Dam. The ground motions were provided under contract to the US Army Engineer District, Sacramento (Bolt and Seed 1983). The peak acceleration of these records is 0.35 g. Two vertical accelerograms were generated from the horizontal components by increasing the frequency content by a factor of 1.5 and by multiplying the amplitudes by 0.6. The response spectra for 5 percent viscous damping computed from the horizontal records are compared in Figure 13. The periods of the first four mode shapes are also shown in Figure 13.

31. Because the monolith is nonsymmetric, the stresses on the upstream and downstream faces are not equal. Accordingly, analyses were performed in which the earthquake forces were applied in opposite directions. That is, the original accelerograms were used (amplitudes times +1) as well as the negative records (amplitudes times -1). Thus, for each earthquake, four different sets of ground motions result: H+V, H-V, -H+V, and -H-V.

32. A total of eight sets of ground motions results when the different combinations of directions are considered for both earthquakes. To determine the critical ground motion, analyses were made using each foundation rock property in Table 4. In these analyses, the material properties of the concrete were an elastic modulus of 5.9×10^6 psi, Poisson's ratio of 0.19, and a unit weight of 158 pcf. In these preliminary analyses, a conservative value of 0.90 was chosen for the wave reflection coefficient α. The results of the analyses are summarized in Table 5 which shows the maximum principal tensile stresses* on the upstream and downstream faces. As shown in Table 5, the absolute maximum stresses (identified by asterisks) for each foundation condition are on the downstream face. For the low foundation modulus, earthquake EQ1, directions H-V, produces a maximum tensile stress of 633 psi. For the intermediate and high moduli, the critical ground motion is EQ2, direction -H+V, and the maximum stresses are 727 and 916 psi, respectively. These stress values are used only for determining the proper earthquake for each foundation since a conservative value of $\alpha = 0.90$ was used for the parameter study. On the basis of these analyses, EQ1 H-V is used for further study for the low modulus foundation, and EQ2 -H+V is used for the intermediate and high moduli.

Response Parameters

33. To ensure the accuracy of the computed dynamic response, the parameters which control the response computations in the program EAGD-84 must be judiciously selected. These parameters are chosen according to the guidelines of Fenves and Chopra 1984.

34. The maximum excitation frequency for which the response of the dam is computed should equal or exceed the frequencies of all of the significant harmonics in the ground acceleration record and the frequency of the highest mode included in the analysis. The digitized earthquake data accurately reproduce ground motion frequencies up to 25 Hz. Because foundation rock flexibility is included in the analysis, 10 generalized coordinates or mode shapes are used to represent the response of the dam. The results indicate

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*Tension stresses are taken as positive.

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that the highest frequency of the tenth mode in any of the dam-foundation systems is 49.1 Hz; accordingly, a maximum excitation frequency of 50 Hz is appropriate.

35. For the specified maximum excitation frequency, the computation of the frequency response functions and the earthquake response is governed by the number of excitation frequencies and the time interval. The number of excitation freqencies used in the analysis is 1024 (2^{10}). For a time interval of 0.01 sec, which corresponds to that of the ground acceleration data, the duration of the response history is 10.24 sec, the frequency increment is 0.049 Hz, and the maximum frequency represented is 50 Hz. The frequency increment of 0.049 Hz is less than 1/50 times the least fundamental natural frequency in any analysis, 4.4 Hz, and thus is sufficiently small to represent the frequency response functions near fundamental resonant peaks. The additional requirements to reduce the aliasing error in the discrete Fourier transform and to ensure accurate computation of the compliance functions for the foundation rock are also satisfied with the values of the response parameters selected as described above.

36. It is worthy to note that for all analyses, the latest time of occurrence of the maximum principal tensile stress is 6.23 sec. It is clear from this result that the 10.24-sec duration for the response history is satisfactory. For these relatively stiff structures, the peak response is expected to occur slightly later than the maximum ground acceleration which, for the horizontal record, occurs at 6.12 sec. Since the horizontal accelerations produce greater seismic effects than the vertical, the result that the maximum stresses occur at 6.23 sec is consistent.

Stress Analysis Results

37. The pertinent stress analysis results are shown in Table 6. For each value of the foundation rock modulus, the associated Poisson's ratio and unit weight from Table 4 are used. The finite element grid used for these analyses is shown in Figure 14. In all cases, the material properties of the dam concrete correspond to those in Table 2. The results in Table 6 show that, as expected, the greatest principal stresses occur for the case in which the foundation modulus and reservoir bottom reflection coefficient are the largest. For this set of parameters, Table 6 shows a maximum principal stress

of 871 psi which occurs on the downstream face at a location of 73.8 ft from the crest. This region corresponds to that at which the vertical downstream face begins its transition to an inclined surface. This stress of 871 psi is greater than the recommended tensile strength of 840 psi for rich concrete. Even though this stress for Case 3 exceeds the tensile strength by 3.9 percent, it is unlikely that extensive cracking will occur. To investigate the depth to which possible cracking might penetrate, contours of envelope values of maximum principal stresses for three cases were prepared as shown in Figures 15-17. Contours of maximum principal stresses at times corresponding to peak stresses in element 120 for Case 3 are shown in Figures 18-21. In order to determine the maximum stress level for Case 3, the envelope of maximum principal stresses is plotted along the upstream and downstream face of the dam, as shown in Figures 22 and 23, respectively. These figures indicate that all compressive stresses are below the ultimate capacity of the concrete. For the worst case only (Figure 17), an area approximately 2 ft in depth on the downstream face is subjected to stresses exceeding 700 psi, the least tensile strength of the dam concrete.

38. The damage to a concrete dam is not predominantly controlled by the transitory peak tensile stress response. Tensile stresses greater than the maximum allowable tensile stress, which are repeated several times during an earthquake, are more damaging than a single large peak stress. In studies of concrete dams subjected to earthquake motion, a maximum repeatable stress level is defined as the maximum stress value that is exceeded by six short duration excursions (US Army Engineer District, Sacramento 1986). Figures 22 and 23 indicate that the maximum principal stresses occur at elevation 196, which corresponds to element 120. Figure 24 displays the maximum principal stress in element 120 as a function of time for Case 3. This figure indicates that the maximum tensile stress exceeds the recommended tensile strength of 700 psi for lean concrete only once during the entire earthquake, and that the maximum repeatable tensile stress is approximately 390 psi.

39. It is reasonable to conclude, therefore, that cracking will be quite limited in extent and depth of penetration into the monolith. It is also worthy to mention that in all cases, the maximum principal stresses in the region of the heel of the dam, at the upstream dam-foundation interface, were well within the tensile strength limits of Table 2. Therefore, no cracking is to be expected in this location as well.

Stability Analysis

40. A stability analysis for seismic loading conditions was performed in accordance with ETL 1110-2-256 (US Army Corps of Engineers 1981) and is presented in Appendix G. The resistance to sliding on horizontal planes at or above the nonoverflow monolith-foundation contact plane was computed. The analysis performed in accordance with the current Corps criteria indicates that the nonoverflow concrete monolith is safe against overturning and sliding at or above the monolith-foundation contact plane.

Conclusions

41. The results show that for the parameters most likely to represent the conditions of the dam, foundation, and reservoir, little cracking will occur in the concrete portions of Folsom Dam. Even under the most unfavorable conditions, the analyses indicate that cracking will be limited in extent and depth of penetration. Based upon these findings, it is concluded that the dam will maintain its structural integrity during and after a major earthquake. Thus, there will be no sudden loss of the reservoir from these postulated earthquakes.

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Table 1

Summary of Material Properties Test Results

	SPD	USBR	UCB
Modulus of Elasticity, psi $\times 10^6$			
Static	4.18	4.18	5.45
Dynamic	-	4.50	5.95
Poisson's Ratio			
Static	0.17	0.14	0.18
Dynamic	-	0.21	0.20
Splitting Tensile Strength, psi			
Static	483	482	363
Dynamic	-	510	539

Table 2

Concrete Material Properties (Appendix D)

Property	Value
Modulus of Elasticity, Dynamic	5.9×10^6 psi
Poisson's Ratio	0.19
Tensile Strength:	
Rich Concrete	840 psi
Lean Concrete	700 psi

Table 3

Field Log Summary of Rich Concrete Location

Monolith	Length of Rich Concrete Core (ft)	Depth from Surface (ft)	General Location
5	0.8	0 to 0.8	crest
5	1.0	0 to 1	crest
5	8.0	0 to 8	downstream face
7	4.5	0 to 4.5	downstream face
7	1.0	8 to 9	downstream face
8	2.0	0 to 2	elevation 322.5'
9	0.6	0 to 0.6	crest
9	1.0	0 to 1	crest
10	none	-	elevation 322.5'
12	1.0	4 to 5	elevation 294.0'
12	none	-	downstream face
13	3.0	0 to 3	elevation 224.0'
17	none	-	elevation 224.0'
17	1.0	0 to 1	elevation 294.0'
19	5.7 or more	0 to 5.7	elevation 322.5'
19	6.3 or more	0 to 6.3	elevation 224.0'
19	5	0 to 5	flip bucket wall
21	1.2	0 to 1.2	crest
21	1.8	0 to 1.8	crest
21	0.5	0 to 0.5	elevation 294.0'
21	2	0 to 2	downstream face
21	2.9	3.5 to 6.4	downstream face
21	0.7 or more	8.3 to 9	downstream face
24	1	0 to 1	crest
24	2	0 to 2	crest
24	none	-	downstream face
28	1.1	0 to 1.1	crest

Table 4

Foundation Rock Properties (Hynes-Griffin 1987)

Modulus of Elasticity, Dynamic (million psi)	Poisson's Ratio	Unit Weight (pcf)
5.8	0.30	167
7.9	0.25	171
11.0	0.20	174

Table 5

Maximum Principal Stresses Versus Ground Motion

$$\eta_F = \eta_S = 0.10 \text{ and } \alpha = 0.90$$

Foundation Modulus (million psi)	Earthquake	Component Directions	Maximum Principal Stress (psi)	
			Upstream Face	Downstream Face
5.8	EQ1	H+V	367	460
		H−V	473	633*
		−H+V	582	502
		−H−V	437	382
	EQ2	H+V	497	463
		H−V	536	597
		−H+V	573	578
		−H−V	452	508
7.9	EQ1	H+V	464	477
		H−V	558	697
		−H+V	624	600
		−H−V	466	498
	EQ2	H+V	534	545
		H−V	664	685
		−H+V	641	727*
		−H−V	519	555
11.0	EQ1	H+V	720	718
		H−V	749	788
		−H+V	725	816
		−H−V	684	763
	EQ2	H+V	560	607
		H−V	843	771
		−H+V	734	916*
		−H−V	591	574

* Maximum for each foundation modulus.

Table 6

Summary of Maximum Principal Stresses

F = S = 0.10

Case	Foundation Modulus (million psi)	Foundation Poisson's Ratio	Foundation Density (pcf)	Wave Reflection Coefficient, α	Earthquake	Direction	Maximum Principal Stress (psi)
1	5.8	0.30	167	0.75	EQ 1	H-V	585
2	7.9	0.25	171	0.79	EQ 2	-H+V	672
3	11.0	0.20	174	0.82	EQ 2	-H+V	871
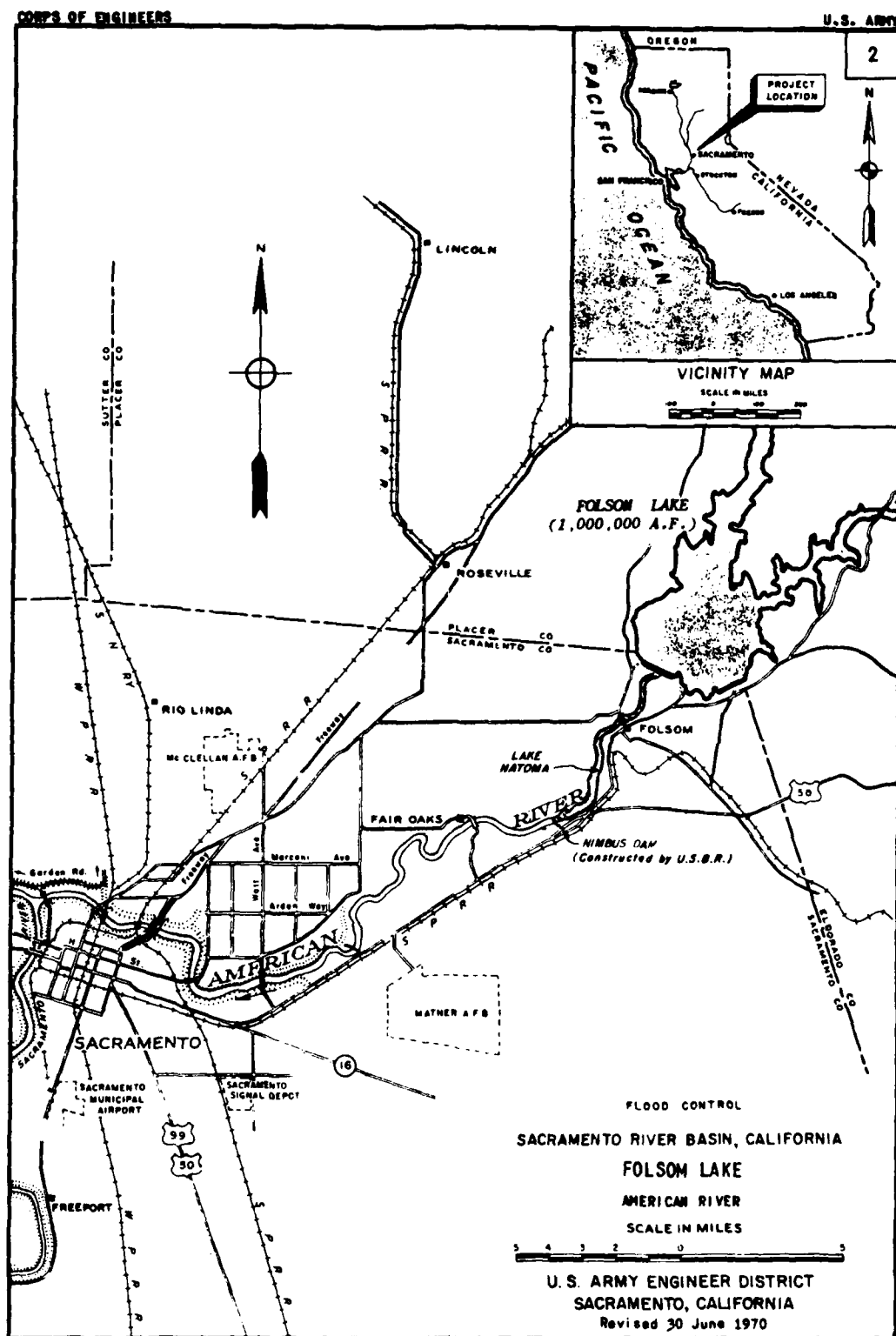
Figure 1. Location of Folsom Dam and Reservoir Project

Figure 2. Seismological study area

Legend:
- Volcanic and unconsolidated rocks
- Metamorphic rocks
- Granitic intrusive rocks

(Modified from Clark, 1964)

CALIFORNIA

Area of Map

Prepared by Tierra Engineering Consultants

Labels on map: Pacific Ocean, Coast Ranges, Great Valley, Sierra Nevada, STUDY AREA, Folsom Reservoir, Sacramento, San Francisco, SCALE IN MILES, 0 50 100, -N-
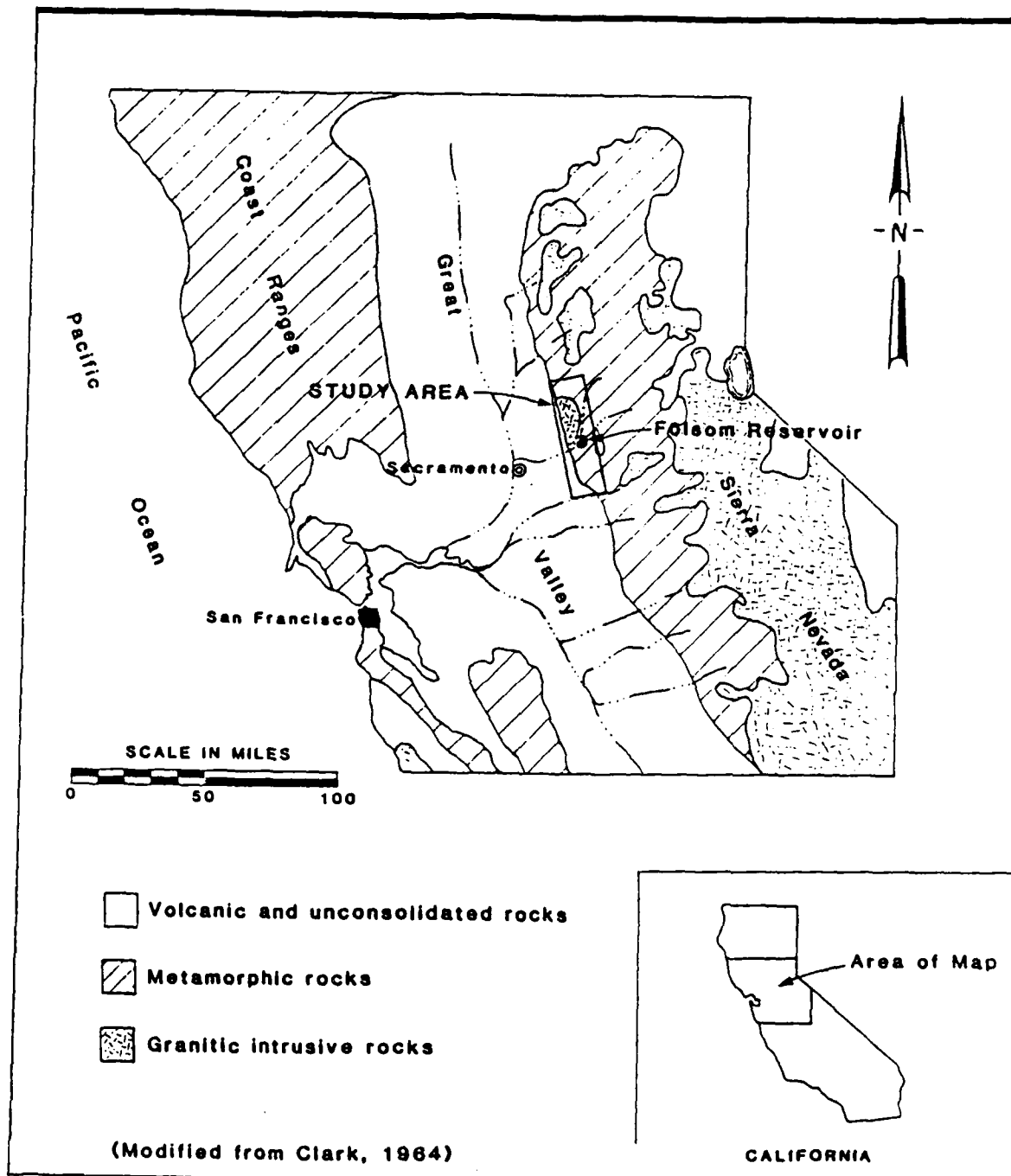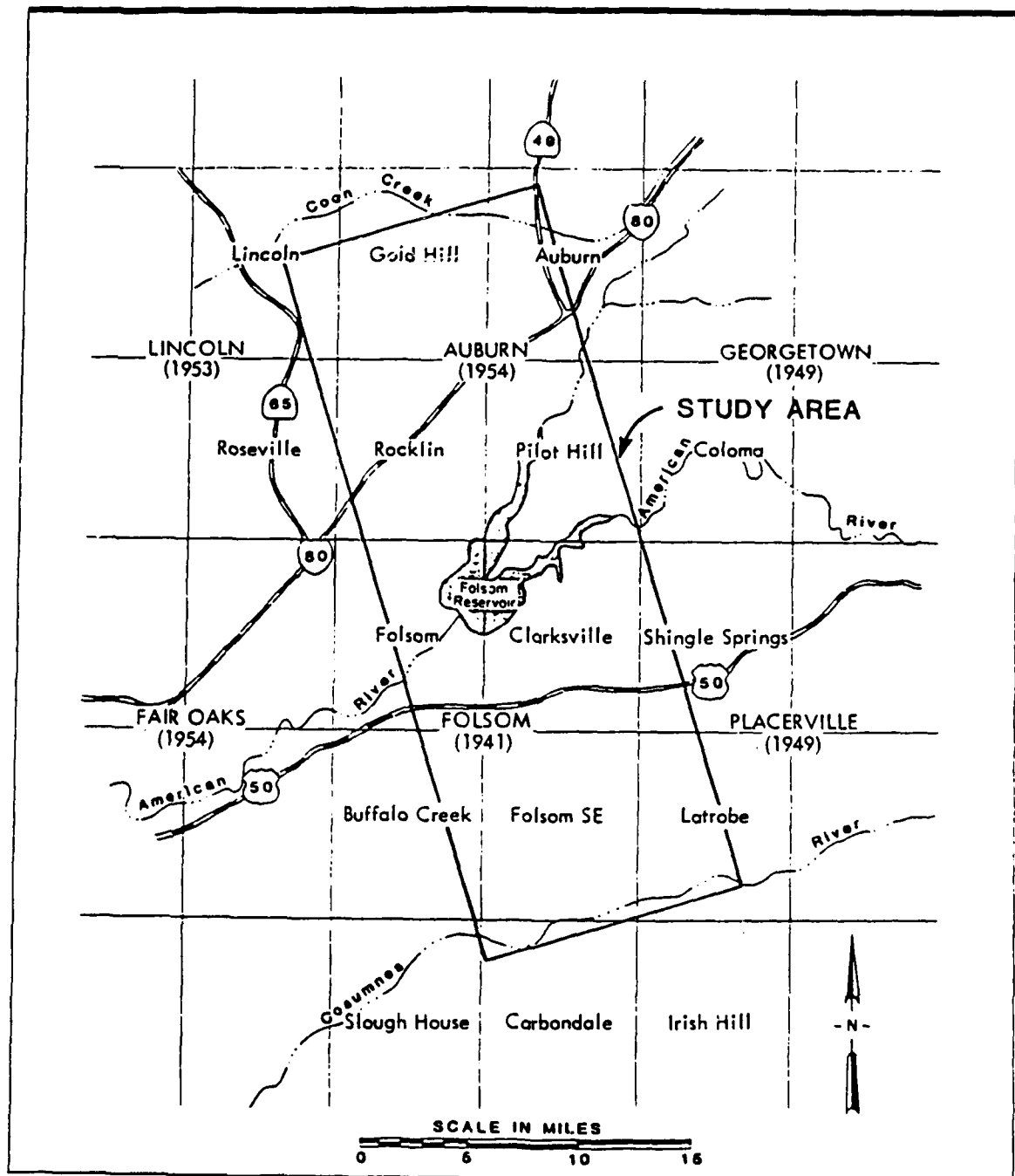
Prepared by Tierra Engineering Consultants

Figure 3. Close-up of study area
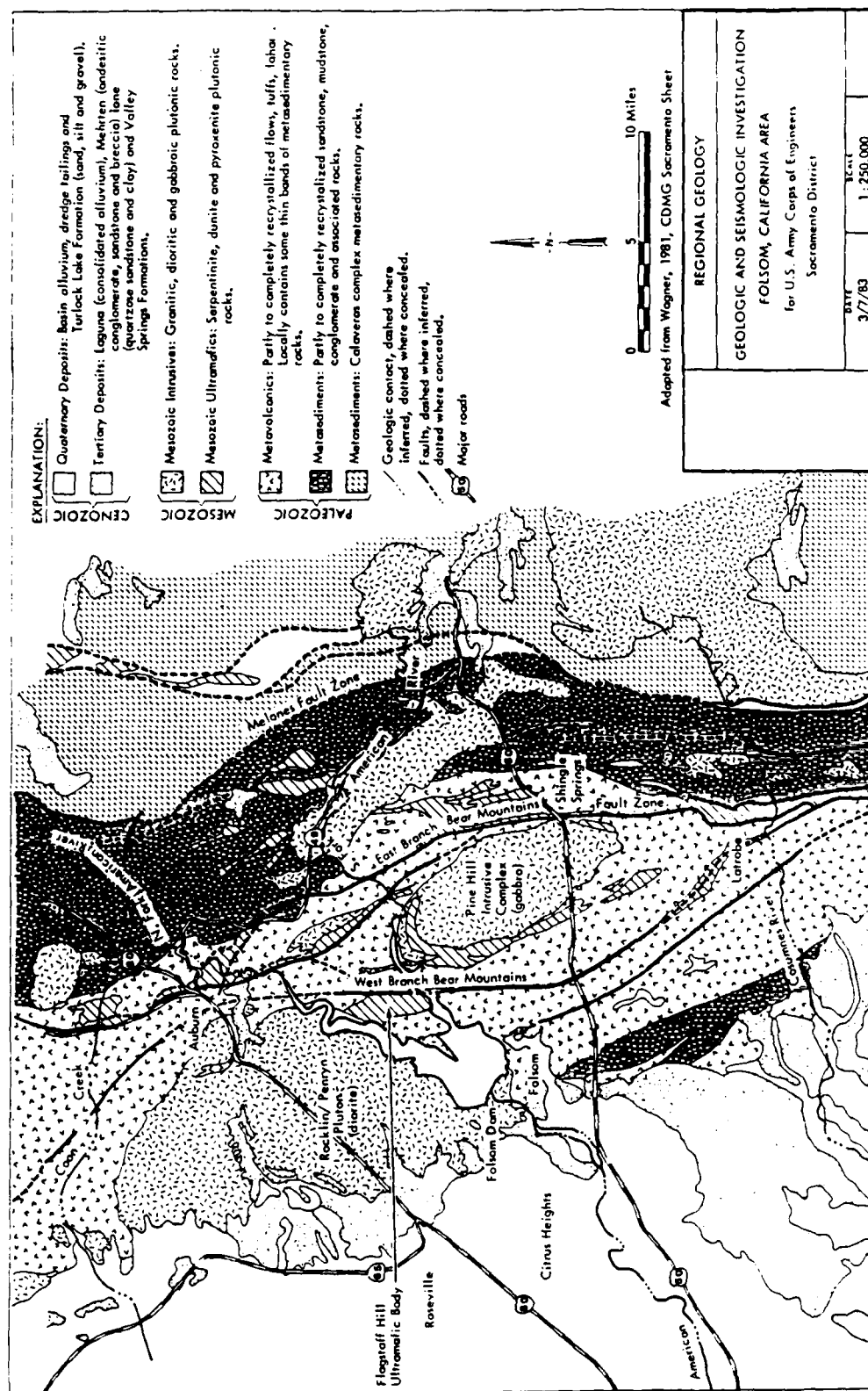
Figure 4. Geologic and seismologic investigation, Folsom, California area

EQ1H

HORIZONAL ACCEL HIST

06/11/67   90630   P1113.40
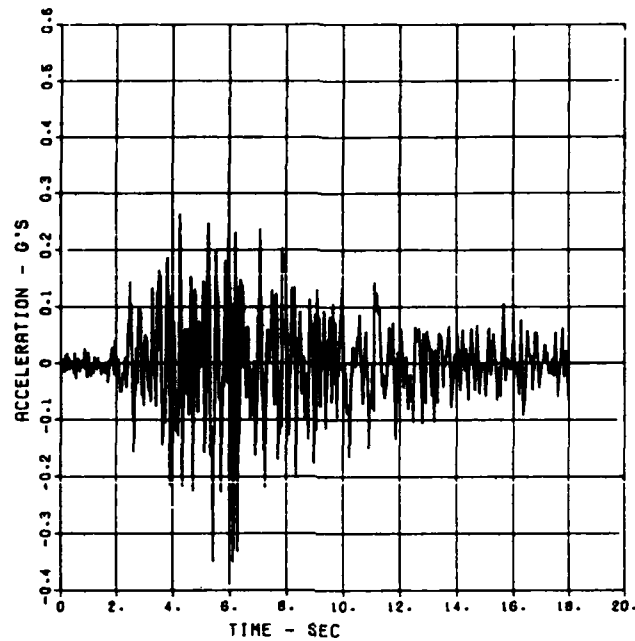
EQ2H

HORIZONAL ACCEL HIST

06/11/67   90630   P1113.40
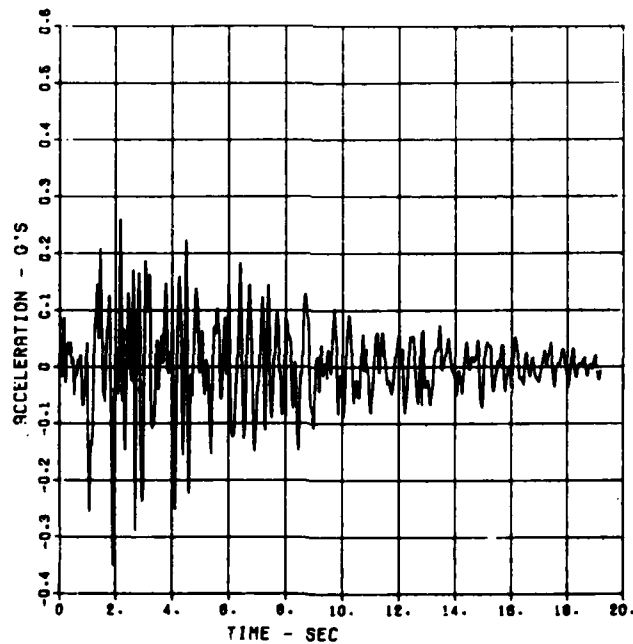
Figure 5. Design accelerograms
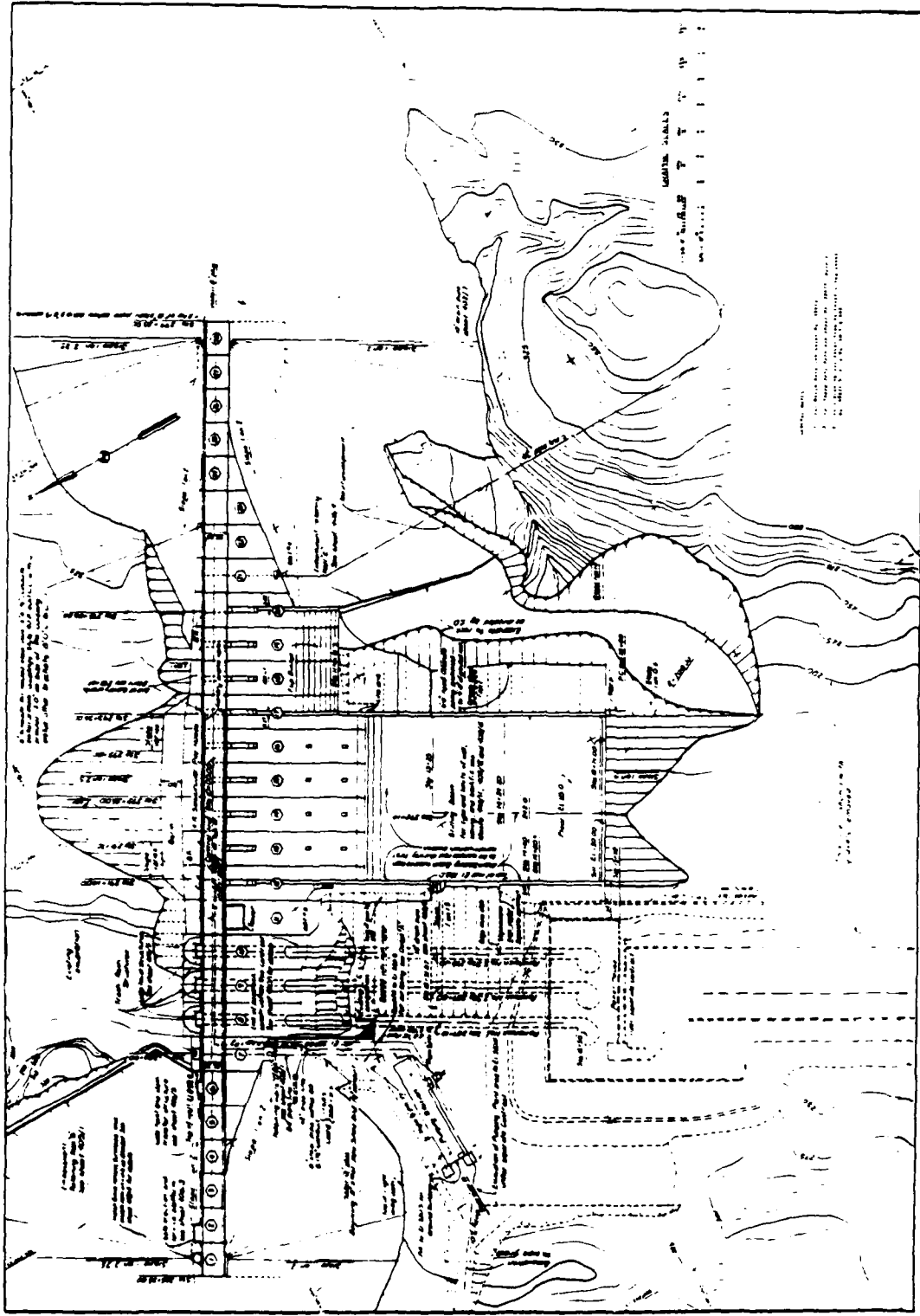
Figure 6. Plan of Concrete Gravity Dam
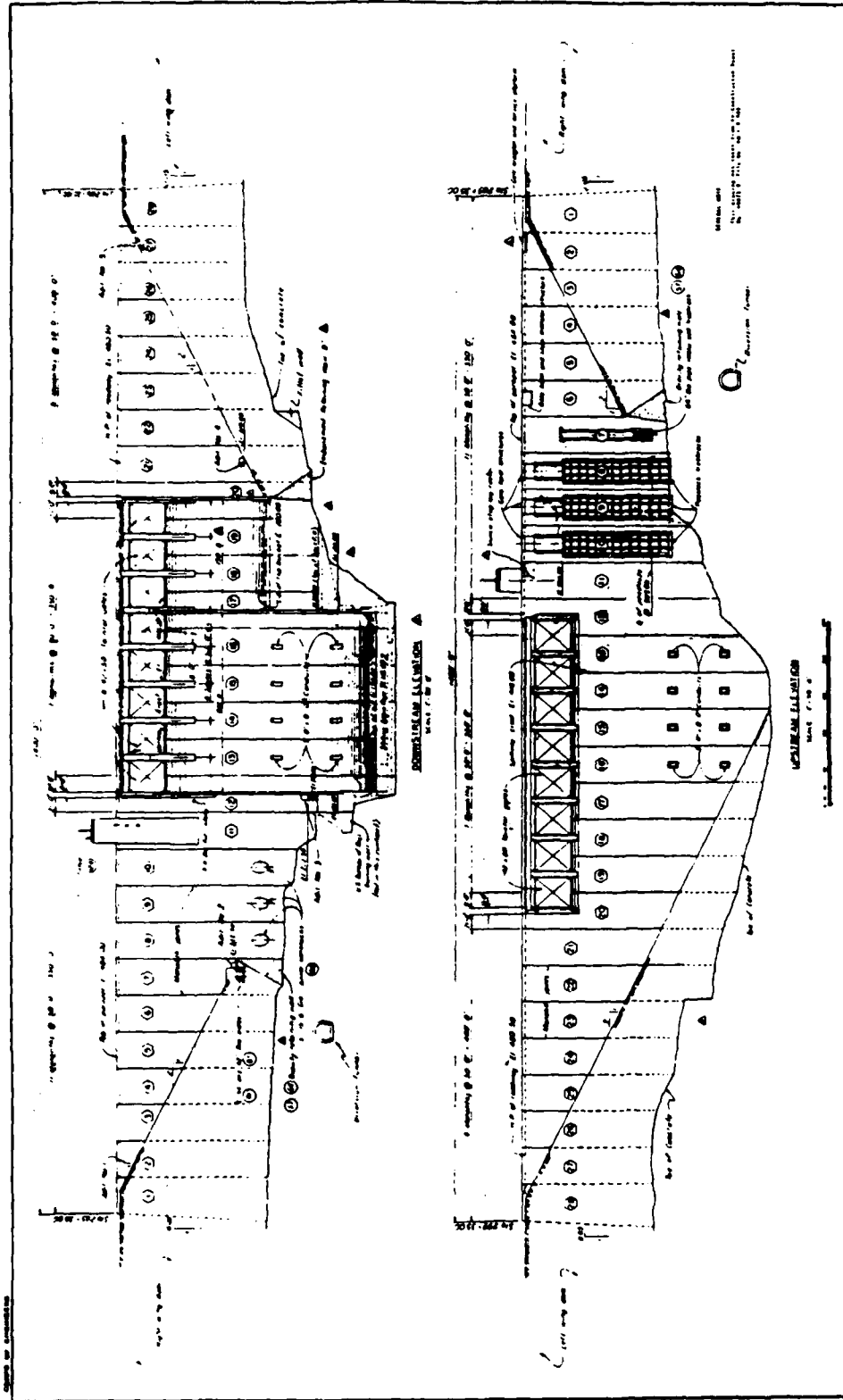
Figure 7. Elevation of Concrete Gravity Dam
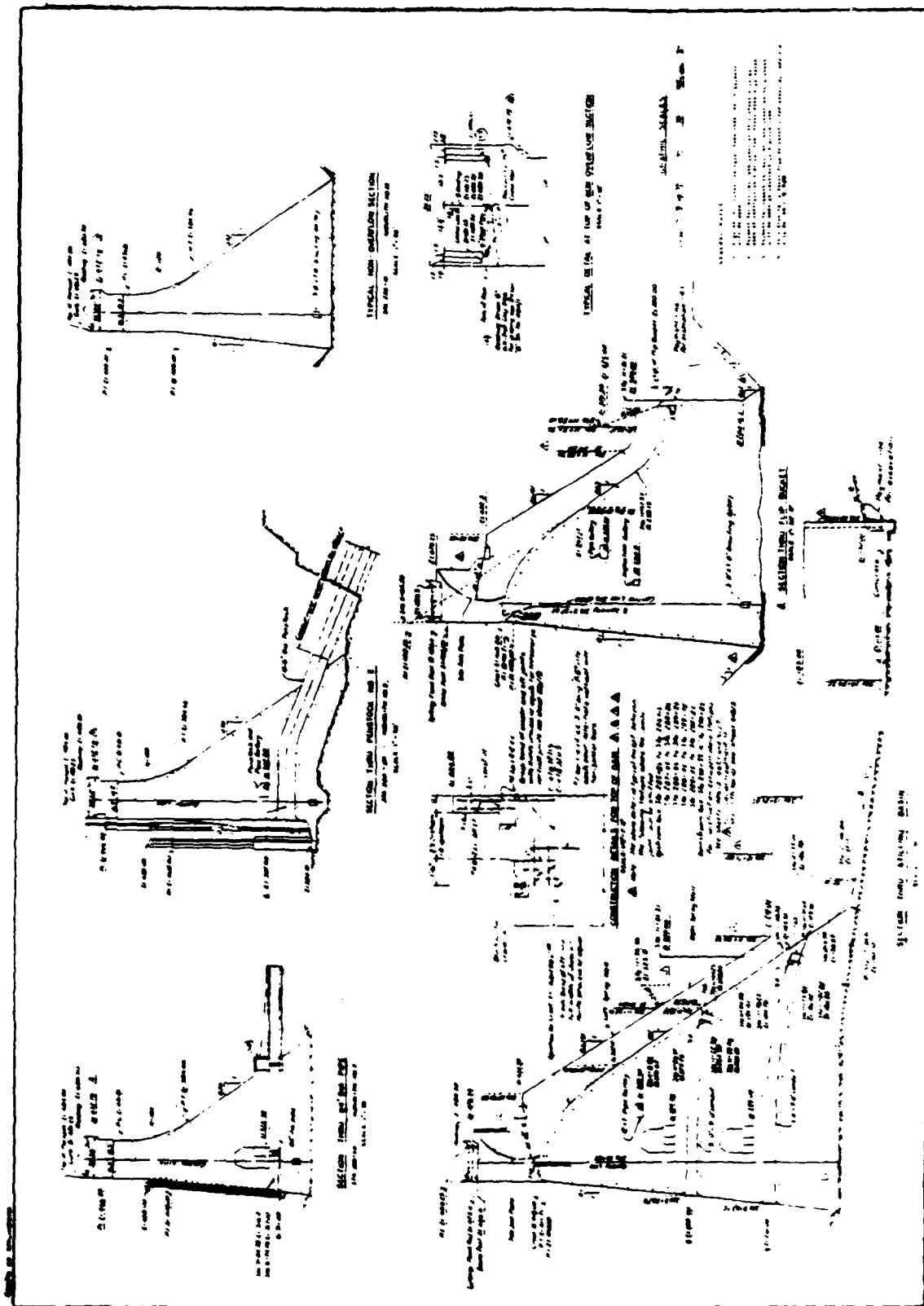
Figure 8. Typical sections of Concrete Gravity Dam

Figure 9. Galleries and shafts in Concrete Gravity Dams

DOWNSTREAM ELEVATION
SCALE: 1'=50'-0'

SECTIONAL ELEVATION

DOWNSTREAM ELEVATION
SCALE: 1"=30'-0"
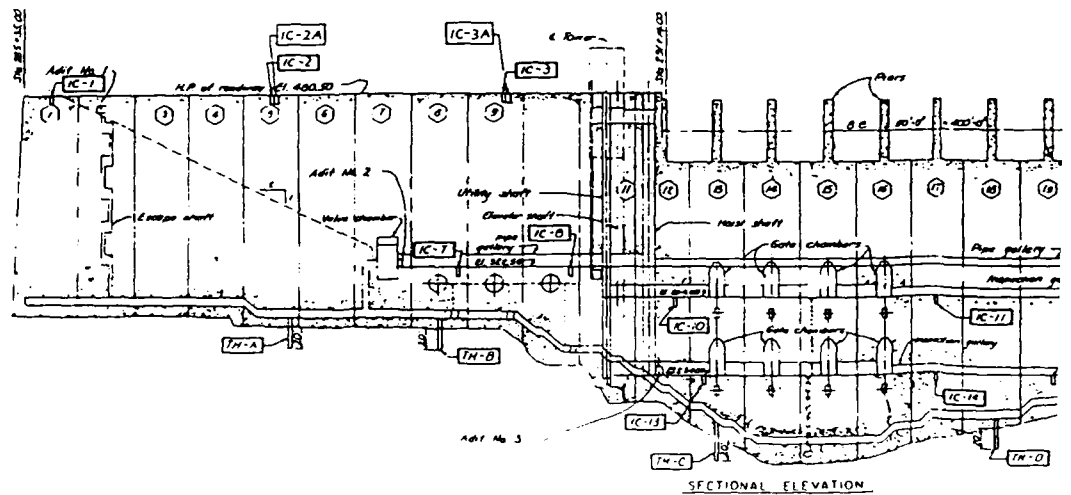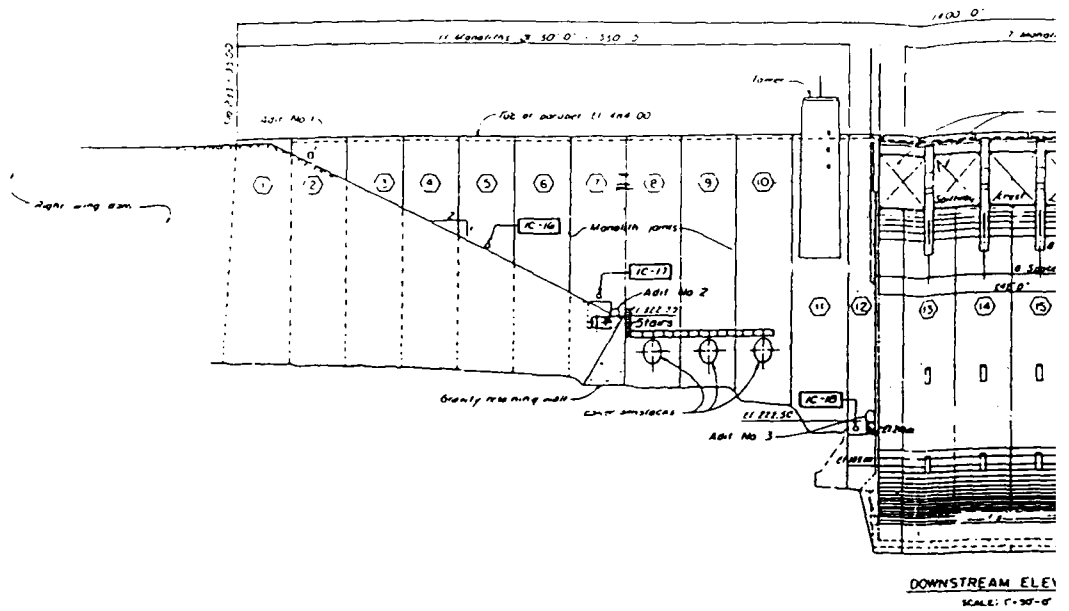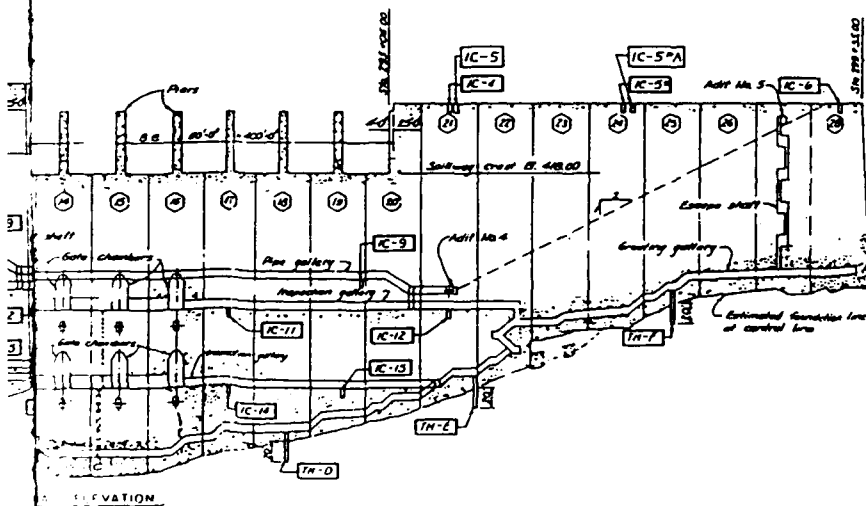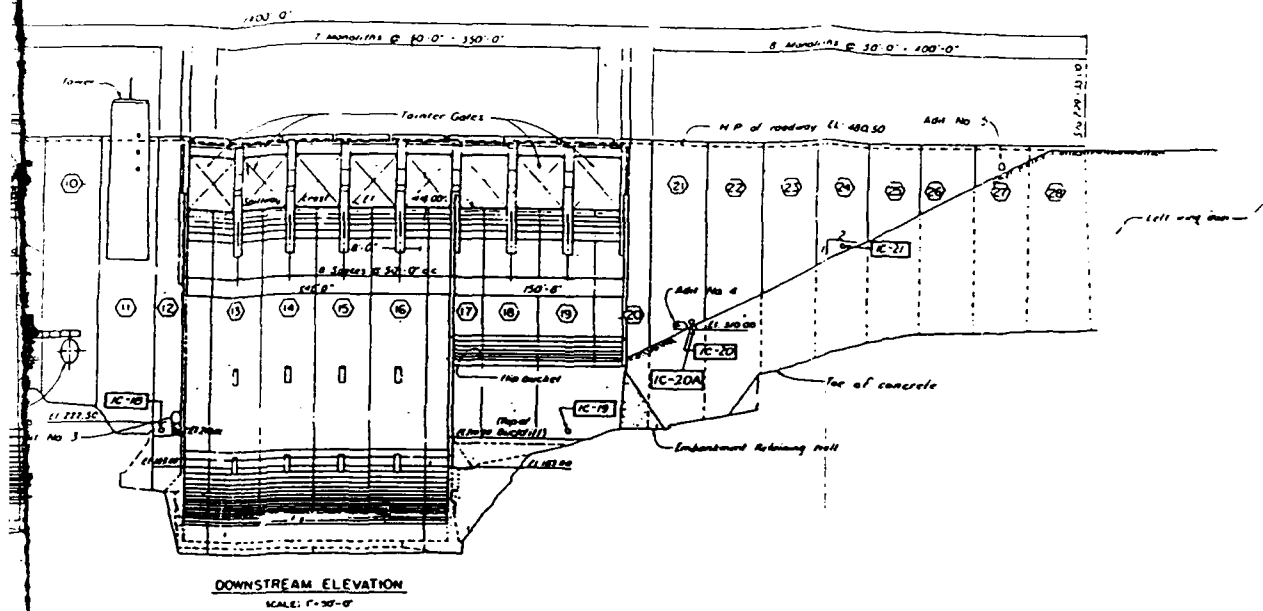
Notes:

1. Concrete Core Drill Sites: IC-18 to IC-21
   Downstream Face

2. Concrete Core Drill Sites: IC-1 to IC-5
   Galleries

3. Rock Core Drill Sites: TH-A to TH-F
   Grout Gallery

GRAPHIC SCALE

ELEVATION

core holes downstream face and sectional views

SECTIONAL

SECTIONAL E.

SECTIONAL PLAN AT EL. 224.00

SECTIONAL PLAN AT EL. 322.50

SECTIONAL PLAN OF GROUTING GALLERY
SCALE 1" = 30'-0"

TH-C   TH-D   TH-E   TH-F

Control line Sta 10+00
Inspection gallery
El. 234.00
Inspection gallery
El. 234.00

SECTIONAL ELEVATION

Domestic water available in Control tower

Piers

IC-5   IC-5*A
IC-4   IC-5*   IC-6

Spillway Crest El. 448.00

Gate chambers
Escape shaft

IC-9

Gravity gallery

Inspection gallery

IC-10   IC-11   IC-12   TH-F
IC-13   IC-13
                 Extended foundation line of control line

TH-C   TH-D   TH-E

Detail "E"
See Dwg A.M.-1-9-405/6)

SECTIONAL PLAN AT EL 294.00

Inspection gallery
Entry shaft
Hoist shaft
Inspection gallery
Control line Sta 10+00

IC-11

Inspection gallery

IC-10   IC-12

Gate chambers

GRAPHIC SCALE

Figure 12. Dam-water-foundation rock system

Figure 13. Horizontal response spectra for 5 percent viscous damping

Figure 14. Grid of tallest nonoverflow monolith

MAXIMUM PRINCIPAL STRESS
EQ1H-V, ALPHA = 0.75
FOUNDATION:
  ER = 5.8
  POISSON'S RATIO = 0.3
  DENSITY = 167 PCF

Figure 15. Envelope values of maximum principal
stresses for Case 1 of Table 6

MAXIMUM PRINCIPAL STRESS
EQ2-H+V, ER = 7.9, ALPHA = 0.79
FOUNDATION:
  POISSON'S RATIO = 0.25
  DENSITY = 171 PCF

Figure 16.   Envelope values of maximum principal
          stresses for Case 2 of Table 6

Figure 17.   Envelope values of maximum principal
stresses for Case 3 of Table 6

TIME = 3.05 SECS

Figure 18.   Contours of maximum principal stresses
for Case 3 at 3.05 sec

TIME = 4.45 SECS

Figure 19. Contours of maximum principal stresses
for Case 3 at 4.45 sec
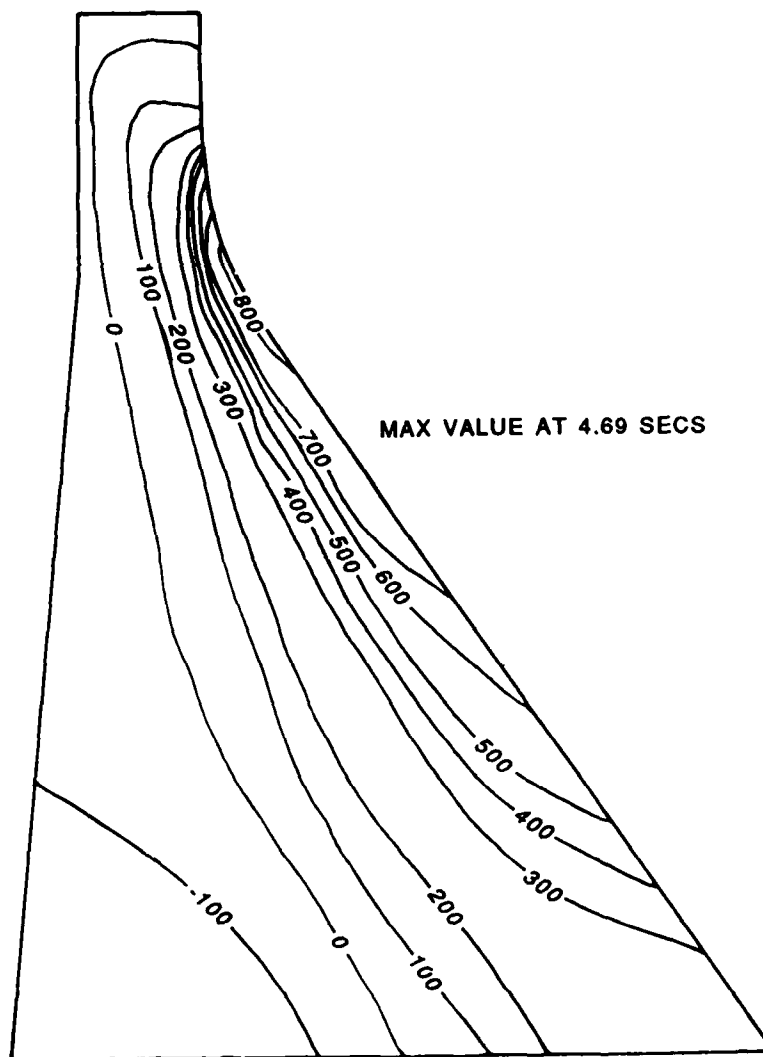
Figure 20. Contours of maximum principal stresses
for Case 3 at 4.69 sec
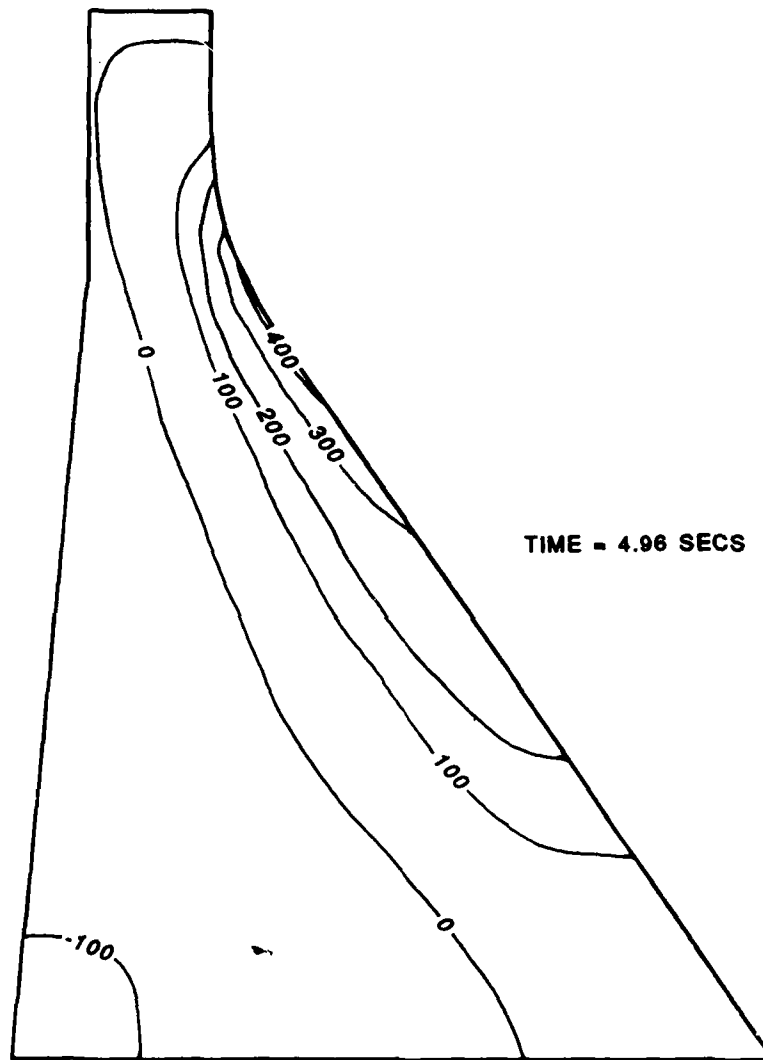
Figure 21. Contours of maximum principal stresses
for Case 3 at 4.96 sec

EQ2-HV,A=.82,ER=11.0
SIGMA-1 UPSTREAM

11090088
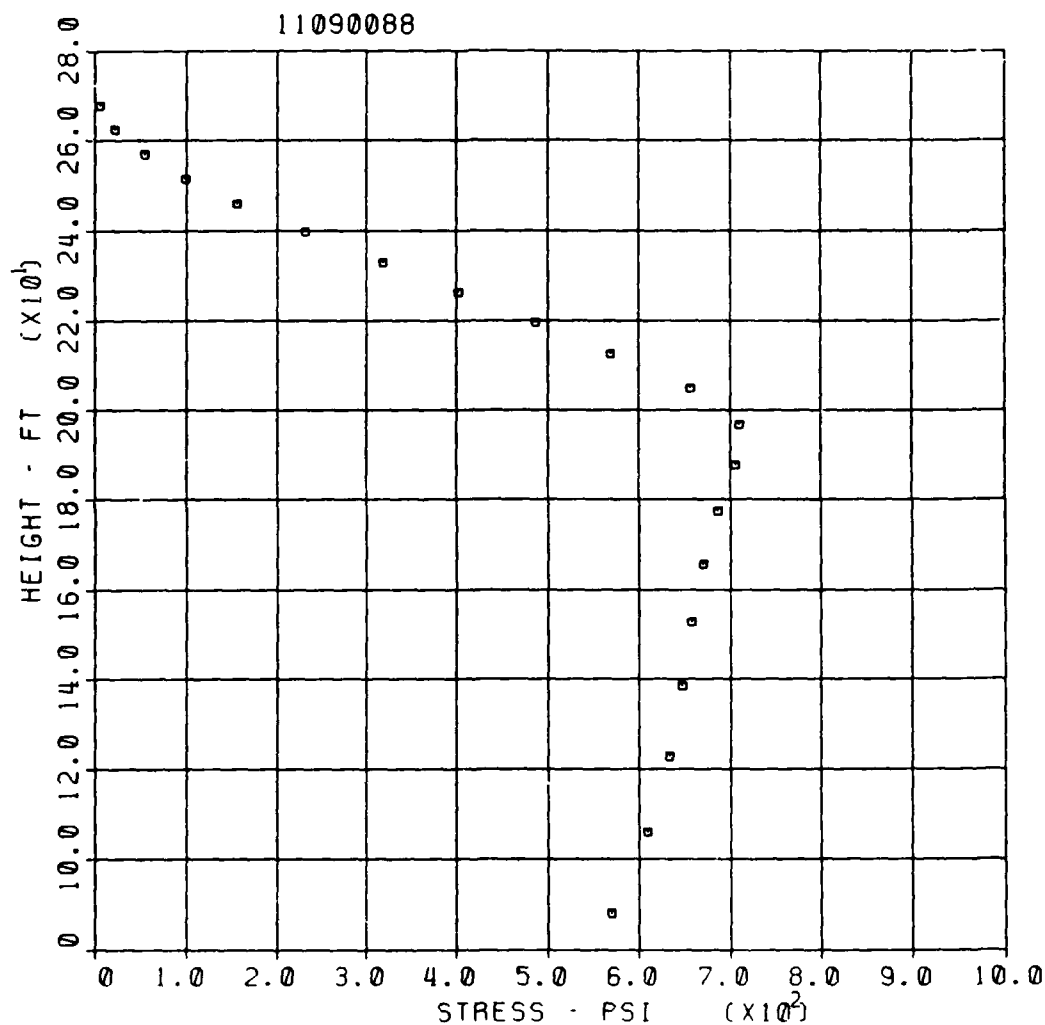
Figure 22. Heightwise distribution of envelope values of
maximum principal stress along the upstream
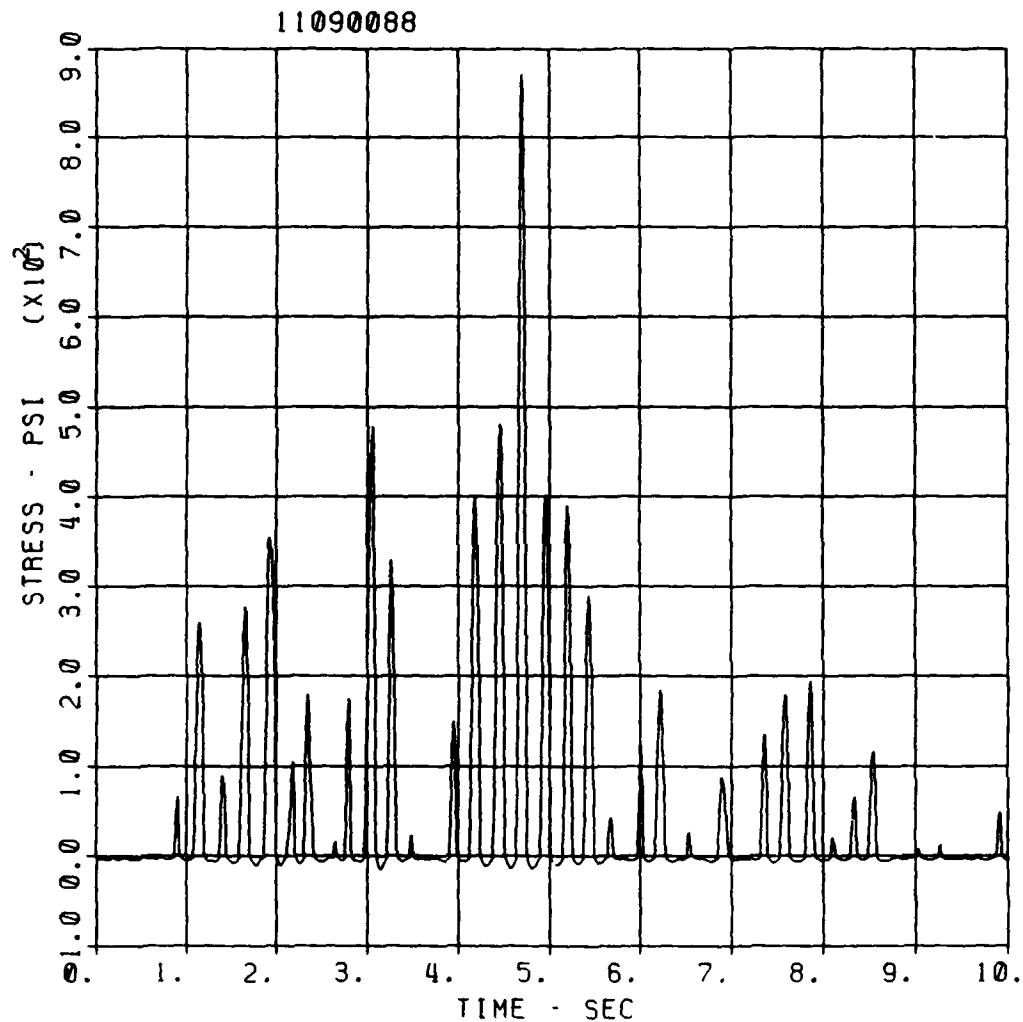face for Case 3

EQ2-HV, A = .82, ER = 11.0
SIGMA-1 DOWNSTREAM

11090088
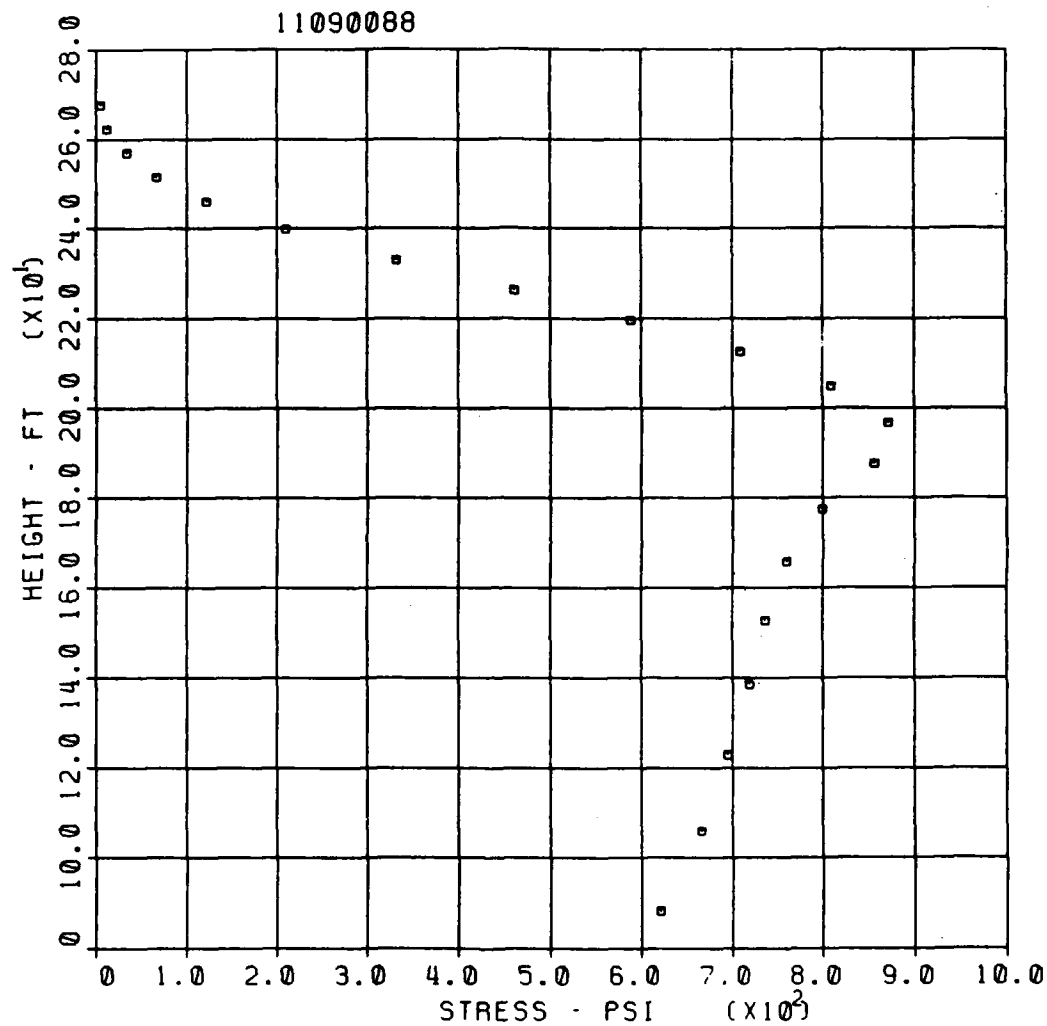
Figure 23. Heightwise distribution of envelope values
of maximum principal stress along the
downstream face for Case 3

EQ2-HV,A=.82,ER=11.0
SIGMA-1 ELE NO. 120

Figure 24. Maximum principal stress in element 120 for Case 3

APPENDIX A

FOUNDATION CONDITION REPORT

FOLSOM DAM PROJECT

by

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February, 1988

PREFACE

This report documents the condition of the foundation materials underlying the Folsom Concrete Gravity Dam. Information contained in this report was accumulated from the foundation reports comprised and presented by the Sacramento District Corps of Engineers. The information presented describes the condition of each of the foundations prior to placement of the 28 Monolithic units that compose the Concrete Gravity Dam. Also presented is a description of the grouting program as it relates to the concrete section of Folsom Dam.

Mr. Glenn A. Nicholson, of the Engineering Geology and Rock Mechanics Division, Geotechnical Laboratory, assisted in the preparation of this report. Mr. Nicholson provided a technical review of the geological engineering aspects of the report and prepared Table 2 which describes structural features exposed in the foundations of the monoliths during construction.

CONTENTS

PART I: INTRODUCTION

Purpose

1. The objective of this report is to summarize information from construction records, engineering geology, sub-surface investigations, excavation and final preparation of the foundation rock underlying the concrete section of Folsom Dam. The Concrete Gravity Dam has a maximum height of 340 ft and a length of 1,400 ft. It consists of 28 monoliths each of which is 50 ft wide. A plan view of the monoliths is shown in Figure 1. As such, this report summarizes the foundation conditions reported in the original foundation reports documenting project construction records (Roddy 1954 a-i). The foundation conditions were ascertained from an extensive geological investigation during the construction phase involving diamond bit drill holes, borehole camera photography, calyx holes, shafts, drifts and trenches. Available records do not document the bases upon which design shear strengths were selected nor do they document initial design analyses of the kinematics of potential slip surfaces.

Description of Foundation Rock

2. The Concrete Gravity Dam is underlain by igneous rock which ranges petrographically from granodiorite to quartz diorite. The rock is medium gray in color and medium to coarse grained. The constituent mineral grains are highly micro-fractured. This granite, of quartz diorite composition, is closely related to and is considered to be part of the Sierra Nevada batholith emplaced during the Jurassic period of time. Extensive weathering has altered much of the foundation rock, which ranges from intensely weathered to essentially fresh.

Main Structural Features of Foundation Rock

3. Four significant faults are located beneath the Concrete Gravity Dam. One of these trends through the Monolith 1 foundation, dips northwestward, and consists of numerous branches and connector fractures. The surface area of the Monoliths 4, 5, and 6 foundations exposed two northwestward

A3

dipping faults. Along each is a zone up to 5 ft wide of breccia, gouge, clay, and chlorite. In the left abutment, a low angle fault was exposed in the foundations of Monoliths 13 to 27; the trend of this fault is N 30° to 45° E, and the dip is northwestward at angles of 20° to 30° below the horizontal.

4. While minor variations existed from monolith to monolith joints could generally be grouped into three major sets. The first and most prominent set strikes N 25° to 60° E and dips 25° to 58° NW; the second set strikes N 80° E to S 80° E and dips 62° to 80° SE-SW; the final set strikes N 0° to 55° W and dips 73° to 87° E-NE. Joints of this set are often either quartz filled or open with rock walls weathered in varying degrees. Of the three sets observed, set one is the most continuous with joint length often exceeding 100 feet. The second set of joints are generally tight but frequently associated with minor shear zones. The last set of joints are generally tight and continuous for only a few 10's of feet.

5. Tables A1 and A2 briefly summarize the foundation conditions below each monolith prior to construction. Specifically, Table 1 summarizes the degrees of rock weathering and geologic features found below each monolith. Table 2 summarizes the orientation joints, faults and shear zones below each monolith.

6. The information contained in Tables A1 and A2 are described in more detail in the following section. The foundation for each monolith is discussed separately. Each discussion presents information obtained from pre-construction and post-construction borings. There then follows a discussion of the structural features of the foundation and foundation conditions prior to concrete placement.

PART II: MONOLITH FOUNDATION CONDITIONS

Monolith 1

7. Monolith 1 is 143 ft long (upstream to downstream) at the base and rises to a height of approximately 200 ft above its foundation. Several holes were drilled into the material beneath Monolith 1 before and during construction to determine a suitable foundation elevation. Before construction, hole 1F-49 was drilled (N 58° E at an angle of 59°45') 42 ft downstream from the toe of Monolith 1. From the surface to el 288.9 persistent zones of intensely weathered granite were present. From el 288.9 to el 259.8 the percentage of intensely weathered granite decreased appreciably. Below el 257.0 the granite core was fresh and the intercepted joints were tight. Three more holes designated 1F-130, 1F-131, and 1F-134 were drilled within the foundation area after some excavation had taken place. All three holes were drilled to approximate el 210. Very little slightly weathered to fresh quality granite was encountered. However, granite which was for the most part moderately weathered and highly jointed was encountered at approximate el 285. It was decided that the foundation grade beneath Monolith 1 should be founded at el. 285.0.

8. Trending through the foundation from NE to SW is a fault zone (strike N 45° E and dip 40° to 62° NW) which consists of a number of irregular branches and interconnectors with the presence of many joints. Due to the presence of extensive amounts of soft, intensely weathered granite on either side of the main branches of the fault, dental excavation was necessary. A V-shaped trench was excavated to reach improved rock. The trench was backfilled with concrete to prevent seepage along this path. Approximately 35 percent of the foundation surface beneath Monolith 1 is moderately to intensely weathered the remaining 65 percent is slightly to moderately weathered. The locations of all drill holes and the fault zone are shown in Figure 2.

Monolith 2

9. Monolith 2 is also 143 ft long at the base and rises approximately 200 ft above its foundation. Three holes designated 1F-107, 1F-109, and

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Calyx 6F-3 were drilled at this location after some excavation had taken place. Hole 1F-107 bottomed at el 212.6 and encountered sound granite at el 281.1. Zones of core loss (representing intense weathering) persisted as deeply as el 258.1. Hole 1F-109 was drilled to el 271.9. Sound granite occurred at el 281.9, below which no core loss was recorded. The third hole, Calyx 6F-3, was a 36 in. shaft drilled near the control line of Monolith 2. The hole was drilled to el 212.7. From the surface to el 246.7 the rock was moderately weathered with alternating open and closed intercepted joints. Below this elevation, the rock quality was essentially fresh and the joints were very tight.

10. Several joints traverse the entire foundation beneath Monolith 2. Most were observed to be less than 35 ft in length, and the degree of openness was not noteworthy. A few major joints, which contain a small shear zone were encountered in this area. The affected rock in the shear zone has been intensely weathered. Most of the major joints trend in a N 88° E direction.

11. The foundation rock at the base of Monolith 2 consists largely (nearly 75 percent) of slightly to moderately weathered granite. The total area of moderately to intensely weathered granite encompasses 750 sq ft (total area of the foundation is 7,150 sq ft). The Calyx 6F-3 hole was backfilled with concrete before construction of the monolith began. The location of the three holes are shown in Figure 3. The foundation grade for Monolith 2 is founded at approximate el 281.

Monolith 3

12. Monolith 3 is 143 ft long at its base and rises approximately 200 ft above its foundation. Two holes were drilled in the foundation for Monolith 3. They are designated 1F-50 (drilled before any excavation and at an angle of 59°41') and 1F-111 (drilled after some excavation). The drill hole locations are shown in Figure 4. Hole 1F-50 was located approximately 8 ft upstream of the control line and penetrated to el 227.0. From the surface to el 280.9 intensely and highly weathered granite was encountered. Below this elevation the granite was sound and the joints were tightly filled with argillaceous material. Hole 1F-111 bottomed at el 264.3. Sound granite predominated below el 300.5. Closely spaced jointing was the most notable structural feature of the Monolith 3 foundation. Granite which ranged from

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fresh to intensely weathered was encountered throughout the area. The ratio of sound to intensely weathered granite is considerably higher for the foundation of Monolith 3 than for the rock beneath Monoliths 1 and 2. Monolith 3 is founded at approximate el 281.

Monolith 4

13. Monolith 4 is 143 ft long at its base and ranges from 212 to 228 ft above its foundation. Several exploration holes were drilled to study the foundation conditions beneath Monolith 4. The location of all holes are shown in Figure 5. Hole 1F-50 (drilled before any excavation) was battered and began in the foundation for Monolith 3 but terminated in the foundation for Monolith 4. The hole was drilled to el 227.0. Above el 280.9, granite that was highly weathered and unsuitable for foundation purposes was encountered. Below this elevation the granite was sound and joints were tightly filled with argillaceous material. Although not shown in any figure, the construction records indicate hole 1F-82 (also drilled before any excavation) was drilled 4 ft upstream of the control line near the Monoliths 3-4 joint. This hole revealed alternating zones of slightly weathered granite cut by open joints and broken, faulted, intensely weathered granite to el 264.1. Below this elevation the core was sound to the base of the hole at el 250.2.

14. Four more holes designated 1F-103, 1F-104, 1F-105, and 1F-106 were drilled in the Monolith 4 area following some initial excavation. Of these four holes only hole 1F-103 will be discussed since it is representative of all holes. This hole was drilled to a bottom elevation of 193.4 ft. Down to el 275.8 the degree of rock weathering was generally intense. From el 275.8 to el 231.8 the core was sound and joints were fairly tight. Between el 231.8 and 227.6 the core was broken and brecciated. This fault zone corresponded to the landward of two northwestward-dipping faults exposed in the Monoliths 5-6 area. This fault is shown in Figure 5. From el 227.6 to the bottom of the hole, the core had many breaks which indicated extensive weathering.

15. In the Monolith 4 foundation, relatively few joints were present. However a prominent shear zone was found near the Monoliths 4-5 joint. Most of the joints as well as the shear zone terminated against a fault, which is present along much of the Monolith 4-5 joint. The fault dips northwestward 30° to 46°, and is filled with fault breccia and brown gerruginous clay in a

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zone up to 2 ft wide. A microdiorite dike, shown in Figure 5, was encountered trending across the foundation beneath the Monoliths 4-5 joint.

16. It was determined that dental excavation was necessary in two areas of the Monolith 4 foundation. One area was near the Monoliths 4-5 joint line where the fault was located. The second area was in a zone of intensely weathered granite that trended east-west through approximately the center of the foundation. To treat both of these areas, a U-shaped trench 55 ft long, 25 ft wide, and 16 ft deep was excavated and backfilled with concrete. Most of the Monolith 4 foundation is smooth and composed of fresh granite, with the exception of the two areas discussed above. The foundation for Monolith 4 is founded at approximate el 278.

Monolith 5

17. Monolith 5 is 155 ft long at the base and rises to heights of 214 to 239 ft above its foundation. The locations of drill holes in this areas are shown in Figure 6. Hole 1F-51 (drilled before excavation riverward at an angle of 57°42'), shown in Figure 7, was located near the toe of the dam along the juncture between Monoliths 5 and 6. Down to el 271.0, the granite was weathered alternating in degree between moderate and intense. Below this elevation the granite was essentially fresh and tightly jointed. Core from hole 207 (drilled before any excavation) from el 333.1 to 252.1 indicated that the granite in this depth interval was extensively weathered.

18. The remaining holes discussed in this section for Monolith 5 were all drilled after some initial excavation. Hole 1F-118 was drilled to bottom el 193.4. This hole also indicated the weathering and openness of joints in this area. It also revealed two northwestward-dipping faults, one of which was encountered in Monolith 4. The location of both faults are shown in Figure 6. With the exception of the landward fault, all the rock beneath el 249.0 was slightly weathered to fresh and the joints were loosely filled with oxidation products. Two core holes designated 1F-140 and 1F-141 were drilled to explore the rock to the invert of the Diversion Tunnel, which crosses beneath the foundation for Monolith 5. In each hole, the granite beneath the landward of the two faults (below el 249.0) was slightly weathered to fresh and the joints were tightly filled with chloritic material.

19. Due to the extensive presence of highly weathered rock in this area, it was decided that a large-size exploration shaft was needed. This shaft was 10 ft by 10 ft, designated 4F-56 and terminated at el 215.0. It was determined from study of this shaft that an extra 10 ft of excavation was necessary to remove the soft brecciated rock produced by the fault zones. The location of the Diversion Tunnel and the exploration shaft are shown in Figure 6.

20. Practically all of the exposed foundation for Monolith 5 consisted of broken, strained granite lying between the two faults. Only a few joints were found, and ranged from tight to loosely filled with oxidation products. A zone of intensely weathered granite was encountered near the control line just southeast of the riverward fault. Elsewhere, slightly to moderately weathered granite was exposed. The microdiorite dike can be seen in Figure 6 trending beneath this monolith also. As can be seen on the figure, the dike has been displaced by the riverward fault. The exploration shaft was backfilled with concrete before construction of the monolith began. Due to the dental excavation around the faults, most of the foundation beneath Monolith 5 extends to approximate el 255.

Monolith 6

21. Monolith 6 is 152 ft long at its base and ranges from 209 to 235 ft in height. The location of all holes and other features mentioned in this section are found in Figure 7. As mentioned previously, hole 1F-51 (drilled before any excavation) was drilled to bottom el 225.0. The first 72 ft of granite was broken and intensely weathered. Fresh, tightly jointed granite was found below el 280.5. All remaining holes discussed below were drilled after some initial excavation. Hole 1F-112 explored the granite to el 259.0. Down to el 275.6 highly broken, decomposed rock was revealed. At this elevation the lower of the two previously described northwestward-dipping faults was encountered. Below el 275.6, the weathering was found to be slight with many joints present. The joints had a thin filling of oxidation products.

22. Hole 1F-116 explored the foundation between el 302.5 and 191.5. Persistent weak, weathered zones occurred as deeply as el 255.0, which was chosen as the foundation grade. Below el 255.0 some core loss did occur, indicating that the joints were open and weathered to depths as much as 60 ft

below the foundation grade. Two holes, 1F-138 and 1F-142, explored the gran-
ite to the Diversion Tunnel invert (approximate el 190). Above el 2358.0 the
rock was soft, contained many closely spaced fractures and was not suitable
for foundation purposes. Below el 248.0 the granite was closely jointed. The
joints were filled with argillaceous or chloritic material.

23. In this area the fault is located next to and parallels the Mono-
liths 5-6 joint. Above this fault the granite had many short strain fractures
and was gradational downward to the fault breccia. Below the breccia zone,
the granite was fresh and contained many joints. Once again the microdiorite
dike can be seen trending through the foundation in Figure 6.

24. There were two small zones of intensely weathered granite encoun-
tered beneath Monolith 6. The dental excavation in the foundation of Mono-
lith 5 was extended into the foundation of Monolith 6 to remove the fault
breccia and other weathered rock associated with the fault. Once the excava-
tion of this material was completed, the resulting foundation was determined
to be acceptable. Except for the area near the fault, a foundation grade of
255.0 was selected for Monolith 6.

Monolith 7

25. Monolith 7 is 153 ft long at its base and ranges from 211 to 215 ft
in height. The location of all drill holes and other features in the Mono-
lith 7 foundation are shown in Figure 8. Hole 1F-51 (drilled before any exca-
vation riverward at an angle of 57°42'), was battered and began in the
foundation of Monolith 6 and terminated in the foundation for Monolith 7.
This hole bottomed at el 225.0. Above el 271.0 the granite alternated between
slightly and intensely weathered. Below this elevation, the granite was
essentially fresh and tightly jointed. Hole 1F-52 (also drilled before any
excavation riverward at an angle of 60°07'), bottomed at el 220.0. Decomposed
and slightly weathered granite was encountered to el 280.5. Below el 280.5,
the granite was once again fresh and tightly jointed.

26. Three more holes designated 1F-114, 1F-117, and 1F-143 were drilled
in the foundation areas after some initial excavation had been performed.
Hole 1F-114 was bottomed at el 253.0 and revealed sound granite below
el 267.6. Above this elevation the core loss was severe, indicating fractures
and weathered material. Hole 1F-117 revealed intensely weathered rock to

el 277.9 where slightly weathered to fresh rock was then encountered to bottom el 196.4. Core from hole 1F-143, bottomed at el 243.3, was slightly weathered to fresh granite. In almost all the holes there was a decided improvement in rock quality noted in the reach between el 275 and 270.

27. Joints were quite prominent throughout this foundation. The granite was intensely weathered where the joint spacing was less than 1 ft. Three fractures which show a slight amount of movement were found and are designated on Figure 8 as minor shear zones. A short section of the east-west trending microdiorite dike crossed beneath the upstream corner of Monolith 7. A few zones of intensely to moderately weathered granite were found in the foundation. The largest area occurred near the toe. The exposed foundation rock beneath Monolith 7 had an unusual degree of surface roughness attributable to the sound granite and numerous fractures. The foundation for this monolith was founded at approximate el 270.

Monolith 8

28. Monolith 8 is 160 ft long at its base and rises to heights of 213 to 218 ft above its foundation. The location of all drill holes and other features in this area are shown in Figure 9. Two holes were drilled in the foundation after some excavation had occurred, 1F-113 and 1F-115. Hole 1F-113 was drilled to el 252.0 and had considerable core loss to el 288.4. Beneath this elevation the granite was fresh to slightly weathered, with few open joints. Hole 1F-115 was drilled near the control line and bottomed at el 198.3. Above el 278.6 some core loss occurred indicating some amount of weathering. Below el 278.6, fresh to slightly weathered granite with few open joints was again encountered. This granite was of exceptionally sound quality.

29. Once again, to prove the adequacy of the right abutment foundation, a large-size hole was drilled. This shaft was designated Calyx 6F-4, a 36-in. diameter boring located near the control line. This hole was drilled after some excavation had occurred, and extended from el 268.0 to 198.2. The granite observed in the shaft was moderately to slightly weathered down to el 242.0. Below this elevation the granite was fresh and quite sound. The foundation was found to have four prominent shear fractures and several joints bordered by strained and decomposed granite. Localized areas of intensely

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weathered granite were found along strain areas near the intersections of several joints. Approximately 80 percent of the granite exposed in the Monolith 8 foundation was fresh to slightly weathered. Before construction of the Monolith began, the Calyx 6F-4 boring was backfilled with concrete. The foundation grade for this Monolith was at approximate el 267.

Monolith 9

30. Monolith 9 is 165 ft long at its base and ranges from 214 to 229 ft in height. The location of all drill holes and other features pertinent to this foundation are shown in Figure 10. Four holes were drilled into the foundation prior to any excavation activity, designated 1F-53 (drilled heelward at an angle of 48°35'), 1F-54 (drilled riverward at an angle of 65°17'), 1F-55 (drilled N 45° E at an angle of 47°04'), and 1F-60. No holes were drilled after excavation began. Hole 1F-53 was drilled heelward to a bottom elevation of 241.5 ft. Down to el 267.3, the core was either unrecoverable or was intense weathered granite. From el 267.3 to 250.4, the granite was essentially sound. Below el 250.4 to el 241.5, there was an indication of extensive weathering along a few joints. Hole 1F-54 was drilled to bottom el 186.1, and revealed extensively weathered soft granite down to el 224.6. Below this elevation the granite was unweathered and joints were tight.

31. Hole 1F-55 was drilled to el 249.6 and revealed alternating zones of intensely weathered granite and relatively fresh, sound granite to el 256.8. Below this elevation the granite was fresh to slightly weathered and of excellent foundation quality. Hole 1F-60 explored rock between el 315.9 and el 217.8. From the surface to el 264.4, the material ranged from fresh to intensely weathered granite. Below this elevation the joints were tight and the core was sound.

32. Several joints were found in the foundation for Monolith 9. Some of the joint traces were observed to be continuous for distances of over 100 ft. The majority of the joints were tight and continuous traces were less than 25 ft in length. Three exceptions were identified and are designated in Figure 10 as minor shear zones. Two large zones of intensely weathered granite were associated with these areas. The remaining exposed surface area revealed fresh to moderately weathered granite; the greater portion was slight to fresh. The majority of the foundation was founded at approximate el 268.0.

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Near the toe of the monolith, the foundation was sloped to a minimum elevation of 255.0 ft due to the many fractures in this area.

Monolith 10

33. Monolith 10 is 166 to 180.7 ft long at its base and rises to heights of 224.9 to 258 ft above its foundation. The location of all drill holes and other features pertinent to this area are shown in Figure 11. Five pre-construction drill holes were placed in the foundation area for Monolith 10 and battered riverward. They were designated 1F-54, 1F-56, 1F-57, 1F-58, and 1F-59. The location of sound rock from these holes was interpreted to be located between approximate el 260 and el 240. Upon termination of some excavation, the foundation sloped riverward between el 256 and el 239. At this point twenty-three percussion holes were drilled. Eight encountered sound granite near el 250, seven penetrated sound granite at elevations ranging from 223 to 235 and eight located sound granite between elevations 217 and 230. Two additional holes, designated 1F-119 and 1F-124, were drilled. Hole 1F-119 was drilled to bottom el 191.0, and revealed extensive weathering down to el 211.2. Below this elevation the joints were tightly filled with argillaceous material and the granite was fresh. Hole 1F-124 was drilled from el 240 to el 198.5. Recovery was 100 percent. All joints intercepted were tightly filled with chloritic and argillaceous material.

34. Numerous joints were exposed in this foundation. Some continuous joint traces as long as 150 ft were observed. Nearly all the joints were open and bordered by alteration plate. Some of the joints were open and bordered by granite that was weathered and friable in nature. Three prominent fractures that had indications of slight movement were identified and are designated as shear zones in Figure 11. In the heel areas of the Monolith 10 foundation, a body of decomposed granite elongated in a direction parallel to the contraction joint, and measuring 25 ft wide and 7 ft deep was encountered. This area, believed to be a result of the shear zones, was excavated to sound foundation rock. Approximately 35 percent of the exposed Monolith 10 foundation was weathered in degrees ranging from slight to intense. The numerous open joints were plugged by the grouting program described in Part III. During dental excavation of the decomposed granite, hole 1F-58 discharged a

sizeable quantity of seepage water and was later plugged. A foundation grade
of approximate el 250 was selected for Monolith 10.

Monolith 11

35. Monolith 11 is 180.7 to 201.4 ft in length and rises to heights of
248.5 to 282.1 ft above its foundation. The location of all drill holes and
other pertinent features of this area are shown in Figure 12. There were no
pre-construction holes drilled into the Monolith 11 foundation. All drilling
discussed in this section was performed after some initial excavation. From
several percussion holes, sound rock was encountered at elevations ranging
from 210 along the Monoliths 11-12 joint line to el 230 along the Mono-
liths 10-11 joint line. Drilled near the control line, hole 1F-125 was bot-
tomed at el 134.2. Between el 234.4 and 174.4, fresh granite contained at
least four open joints down which weathering agents had descended. Below this
elevation the joints were tightly filled with argillaceous material.
Hole 1F-126 was bottomed at el 191.6, and revealed numerous joints widely
spaced and weathered.

36. Beneath the foundation of Monolith 11, an unusual number of N 45° E
joints were bounded by alteration plate and open as much as 0.2-0.6 ft. The
most notable of which followed the Monoliths 11-12 contraction joint from the
heel to the toe. There were also two small fractures (dipping N 88° E) show-
ing a minor amount of movement and open as much as 0.2 ft. An estimated
70 percent of the surface granite in the Monolith 11 foundation is slightly
weathered to fresh. The remaining 30 percent of the area being six patches of
intensely to moderately weathered granite. Some minor dental excavation was
required to remove loose granite in these weathered areas and was later back-
filled with concrete. The foundation for Monolith 11 is founded at approxi-
mate el 225.

Monolith 12

37. Monolith 12 is 201.4 to 260.7 ft long and rises 276.2 to 300.8 ft
above its foundation. The location of all drill holes and other pertinent
features of this area are shown in Figure 13. One boring, designated 206, was
placed in this area prior to construction. Sound foundation rock was

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encountered at el 203.7. Two holes, designated 1F-144 and 1F-205, were
drilled into the foundation after initial excavation had occurred. Hole 1F-
144 was located between el 212.9 and el 166.8. Down to el 189.9 the granite
was intensely weathered, with several open joints filled with meteoric quartz.
Below el 189.9, the granite was fresh and joints were tightly filled with
argillaceous material. Hole 1F-205 explored the foundation from el 200 to
el 73. Down to el 161.0 the intercepted joints were open and the rock adja-
cent to such joints was lightly weathered. Below el 161.0 all joints were
tight or had a compact filling of chlorite and clay. Between el 104 and
el 101 the granite was dark-colored, fine textured and showed evidence of
recrystallization of the primary minerals; this was a fault zone. The central
fracture of the fault zone was located at el 103, was open 1/2-in., and the
bordering rock was ragged, soft and affected by numerous irregularly-trending
strain fractures. Bore hole photographs of the fracture were made and are on
file.

38. Several joints were present that were slickensided and line with
gouge, bordered by alteration plate and locally filled with meteoric quartz.
Only one fracture traverses the entire width of the Monolith 12 foundation,
and is designated as a shear zone. It bears the same characteristics as the
joints described above. Near the toe, a trace of the minor low-angle fault,
which was extensively exposed as far up as the Monolith 27 foundation, crosses
the Monoliths 12-13 joint line. The walls of the fracture show varying
degrees of chloritization affects. The extent and intensity of weathering
decreases measurably from that which affects the Monoliths 10 and 11 founda-
tions. Fewer joints were open, and where open joints did occur weathering was
reduced. Fresh to slightly weathered granite comprises approximately 90 per-
cent of the surface granite beneath Monolith 12. The rest of the granite is
slightly to moderately weathered reflecting the diminishing influence of the
joints. In the Monolith 12 foundation, it was necessary to excavate approxi-
mately one-half of the material an extra amount to clear the overflow section
of the Concrete Gravity Dam. This marks the beginning of the stilling basin
area. Because of this, half of the foundation is founded at approximate
el 200 while the other half is founded at el 140. Also found in the founda-
tion of Monolith 12, are seventeen anchor bars 35 ft in length.

Monolith 13

39.   Monolith 13 is 260.7 to 264.7 ft in length and rises 298.5 to
333.7 ft above its foundation.   The location of all drill holes and other per-
tinent features of this area are shown in Figure 14.   Five pre-construction
holes were battered designated 1F-65, 1F-86, 1F-88, 1F-91, and 1F-93.
Hole 1F-65 was bottomed at el 127.8, with no core being taken until el 154.2.
Below el 154.2, sound fresh granite with tightly filled joints was revealed.
Drilled to bottom el 121.8, hole 1F-86 encountered fresh granite at 149.2.
This hole also detected a quartz filled fault zone between el 147.3 and
el 146.8.   Hole 1F-88 detected fresh granite with tightly filled joints at
el 176.5, and was bottomed at el 146.0.   Slightly weathered to fresh granite
was detected at el 197.3, with the joints again being tightly filled.
Hole 1F-93, bottomed at el 144.5, encountered fresh granite at el 175.0.
There were no holes drilled into the foundation post-construction.   This was
decided because earlier exploration indicated that most of the weathered rock
in the stream channel had been removed by erosion prior to deposition of the
channel gravel.

40.   An unusually large amount of joints were present and affected by
linear exfoliation.   Erosion occurring prior to the deposition of the channel
gravel had removed almost all the weathered granite adjacent to these joints.
Trending across the heelward portion of the foundation was the same fault
encountered in the foundation of Monolith 12.   The central fracture of the low
angle fault had the same characteristics as before.   Several smaller, less
pronounced fractures radiate out from the main fracture.   Most reveal normal
movement; at other localities the movement is reverse.   Weathering had pro-
gressed farthest along the series of closely spaced joints located heelward of
the control line.   Two relatively small areas of slightly to moderately weath-
ered granite can be seen in Figure 14.   Slightly weathered to fresh tightly
jointed granite, which is for the most part stream polished, occurs beneath
the remaining portions of Monolith 13.   Fifty-seven anchor bars, 35 ft long,
are placed into the toe area of the foundation.   The foundation is founded at
el 190 near the heel and slopes down to el 130 near the toe.

Monolith 14

41. Monolith 14 is 265 ft long at its base and reaches a maximum height of 343.8 ft above its foundation. This monolith is included in the gated overflow spillway section of Folsom Dam. The location of all drill holes and other pertinent features of this area are presented in Figure 15. Three pre-construction holes designated 205, 1F-69, and 1F-94 were placed in this area. These holes explored depths of granite between el 205 and el 4.0. Gravel deposits resting upon granite ranged in depth from 52 to 63 ft. With the exception of a fault zone, which will be discussed below, the granite explored was tightly jointed and of an essentially sound quality. Hole 205 penetrated the fault zone between elevations 130.5 and 132.4, showing soft, extensively weathered granite. The same fault zone occurs in hole 1F-94 between eleva-tions 128.0 and 129.8. No construction exploration was planned nor performed for the foundations of the river channel monoliths, of which Monolith 14 belongs.

42. Trending nearly normal to the control line and dipping riverward, the low angle fault was the most prominent structural feature of this area. Throughout the above defined extent the fault consisted of one major fracture zone, numerous short fractures, soft chlorite and thin lenses of broken mete-oric quartz. Above the fault zone joints occurred that showed linear exfoli-ation and were bordered by alteration plate. Below the fault joints were widely spaced containing a thin, tight filling of light blue siliceous clay or chlorite. In eroding its channel the American River removed most of the decomposed granite except for a narrow strip near the Monoliths 13-14 contrac-tion joint. Much of the fresh granite was stream polished, pot-holed, and tightly joints. Numerous of the joints occurring near the heel and affecting granite above the fault plane were bounded by minor quantities of weathered material. Since Monolith 14 is in the spillway section, an extra amount of excavation was performed in order to reach granite of quality necessary to comply with design subgrade. Fifty steel anchor bars were then placed in the toe area of the foundation. The Monolith 14 foundation exposes a stream polished, smooth, considerably pot-holed surface having practically no relief. Below the fault trace the joints were tight and unweathered; in the granite lying above the fault zone joints were locally altered, weathered or open, but the intervening granite blocks were quite sound. Beneath the toe section,

excavation to sound rock left the surface sharply irregular. The foundation is founded at approximate el 140.

Monolith 15

43. Monolith 15 is 265 ft long at its base and reaches a maximum height of 343.8 ft above its foundation. This monolith is included in the gated overflow spillway section of Folsom Dam. The location of all drill holes and other pertinent features of this foundation are shown in Figure 16. Vertical core holes 1F-64, 1F-67, 1F-89, and 1F-95 were battered into the Monolith 15 foundation. Extending 19 to 38 ft into foundation granite, all the core holes penetrated sound, tightly jointed granite below a weathered fault zone. In hole 1F-67 the fault zone was encountered between elevations 143.5 and 144.6 and in hole 1F-95 between elevations 144.3 and 153.2. Holes 1F-64 and IF-67 were drilled into sound granite located below the fault plane. A fifth hole, designated 1F-11, was collared in the base of a shallow shaft located on the right abutment and drilled beneath the river. From this hole the low angle fault was represented by altered, fine textured and chloritized granite bounding an irregular central fracture. No construction exploration was planned or executed for this monolith. On the prepared surface of the Monolith 15 foundation the fault trace was all exposed heelward of the control line. Prior to excavation of this area, the material was decomposed so extensively that the resulting material was a soft, saturated, loosely-bonded combination of meteoric quartz, iron stained clay, fault gouge and decomposed granite.

44. Joints were present being more numerous below the fault than above, but were shorter in extent. None of the joints occurring beneath the fault exhibited linear exfoliation; instead they were generally filled with light blue, siliceous clay. A line paralleling the Monolith 14-15 contraction joint and 5 to 12 ft inside Monolith 15 marks the edge of fresh, tightly jointed, pot-holed granite. Two relatively small areas totaling about 700 sq ft, are designated on Figure 16 as moderately to intensely weathered granite. There is a large area, paralleling the Monoliths 15-16 joint line, that composes the fault. Here the granite is essentially fresh capped by tabular shaped plates of altered granite which grade downward into fresh granite. Locally the plates were loose or drummy and the joints beneath were completely obscured. Thirty anchor bars, 21.4 ft in length, were embedded into the rock beneath the

toe. The majority of this foundation is founded at el 140 with the toe section sloping to as low as el 130.

Monolith 16

45. Monolith 16 ranges from 261 to 263 ft in base length, and rises to a maximum height of 335.2 ft above its foundation. This monolith is included in the gated overflow spillway section of Folsom Dam. The location of all drill holes and other pertinent features of this foundation are shown in Figure 17. Vertical core holes 1F-66, 1F-68, 1F-87, 1F-90, 1F-92, and 1F-96 were battered into the Monolith 16 foundation. Similar to the Monolith 15 foundation granite, the rock cored beneath the low angle fault was tightly jointed and quite sound. Through the fault zone weathering ranged from intense to slight and much core loss was observed. The faulted zone was located by hole 1F-66 between el 161.2 and 182.8, by hole 1F-68 between el 147.0 and 149.6, by hole 1F-87 between el 177.9 and 188.1, by hole 1F-90 between el 167.7 and 169.2, by hole 1F-92 between el 161.9 and 167.4 and by hole 1F-96 between el 153.4 and 158.0. No construction exploration was planned or performed in this foundation.

46. Approximately 90 percent of the foundation beneath Monolith 16 revealed the fault surface. Before excavation of the fault zone, the material consisted of a saturated mass of quartz, clay, and decomposed granite. All the weathered material was excavated, with sizeable quantities of water discharge being evidenced from various levels of the fault zone. After placement of the grout curtain, discussed in Section III, no significant discharge was evidenced. Joints were few in number and spaced several feet apart; all were tight and unweathered, or tightly filled with thin layers of siliceous clay. All the foundation granite, left after excavation, is relatively free from weathering. Practically all the surface heelward is the alteration coated fault footwall. The foundation is founded at approximate el 145, except for that portion near the toe which dips to el 132.

Monolith 17

47. Monolith 17 ranges in base length from 254 to 263 ft, and in its height above foundation from 314.1 to 324.2 ft. This monolith is included in

the gated overflow spillway section. The location of all drill holes and other pertinent features of this area are shown in Figure 18. Pre-construction hole 1F-11 (drilled beneath the river at an angle of 30°) indicated sound granite with tight joints, and intercepted the low angle fault zone between elevations 121.8 and 121.0. This is the same fault located beneath Monoliths 14, 15, and 16 with characteristics as described therein. After construction had begun, hole 1F-53 was drilled to bottom el 148.5. Between elevations 167.4 and 168.5 the low angle fault was encountered. Below the fault zone the granite was chloritized, fine textures, and dark green in color. Below this the granite graded imperceptibly into fresh, tightly jointed rock. Hole 1F-54 was also drilled, and revealed results very similar to those of hole 1F-53. The above mentioned fault traced parallel with the Monolith 17 heel.

48. Joints were few in number in this foundation with those present being tight and showing almost no alteration. One joint which crossed the entire width of the foundation showed normal displacement and was designated as a shear zone. With the exception of the fault zone, the foundation consists of fresh to slightly weathered granite. There are two areas designated as slightly to moderately weathered granite which reflect their proximity to weathering agents circulating through the fault zone. In the fault area approximately two feet of additional excavation was required to remove the weathered material.

49. One-half of the Monolith 17 spillway chute descends the face of the dam to discharge its flow into the stilling basin, the other half discharges over the flip bucket. In order to reach design subgrade an additional 25 ft of the toe area of this foundation was excavated. After consultation with all parties involved, it was agreed that if the rock mass above the fault could be stabilized and thereby prevented from sliding riverward the foundation would be competent. Accordingly, a block of concrete, anchored by steel dowels grouted into the rock below, was placed upon the fault footwall. This retaining block was so aligned that its riverward edge coincided with the Monoliths 17-18 joint line. Sixteen anchor bars 25 ft long were placed in the toe area of this foundation. The entire area of this foundation is irregular and founded at varying elevations. The majority being founded at approximate el 165.

Monolith 18

50. Monolith 18 ranges in its base length from 231 to 254 ft, and in height from 303.0 to 314.1 ft above its foundation. This monolith is included in the gated spillway overflow section. The location of all drill holes and other pertinent features of this area are shown in Figure 19. Prior to any construction two holes designated 1F-44 and 1F-70 (drilled S 2°23' W at and angle of 29°11') were drilled into the foundation material. Hole 1F-44 was located near the heel of Monolith 18 and explored granite to el 135.8. Once again the low angle fault was encountered (El. 176.4 to 175.3) with the same characteristics as previously described. Below this the rock was generally found to be fresh and tightly jointed. Hole 1F-70 presented the same results as hole 1F-44, with the fault being discovered between el 179.9 and 178.2. There were no post-construction holes in this foundation.

51. Joints in this foundation tended to be short with spacing between being greater than 7 ft. Only a minor amount of iron-stained alteration plate affected such joints, with the few exposing plated alteration being quite tight. Tracing the entire width of the foundation was a joint designated as a minor shear zone. This is the same shear zone that was seen crossing Monolith 17, and was composed of a soft, light-green clay. Except for the area affected by the fault, the granite is essentially fresh and tightly jointed. The area of the fault revealed plated, chloritized granite. In this area an extra amount of excavation, averaging 8 ft, was performed.

Monolith 19

52. Monolith 19 ranges from 220 to 231 ft long at its base, and rises 290.8 to 303.0 ft above its foundation. This monolith is included in the gated overflow spillway section. The location of all drill holes and other pertinent features of this area are shown in Figure 20. Hole 1F-45 (drilled N 56°36' E at an angle of 31°39') is the only pre-construction boring that will be discussed, since it adequately portrays the conditions prior to initiation of construction. This hole encountered the fault zone between el 195.0 and 193.8. Below this the granite was tightly jointed and quite sound. Three holes were placed in the foundation after some initial excavation had taken place. Hole 1F-128, located near the control line of

Monolith 19, was cored to a bottom elevation of 165.5. Below the fault zone (El. 189.9 to 188.4) the granite adjacent to joints was chloritized and locally soft; however, all the joints were relatively tight. Hole 1F-190 was drilled from el 247.4 to el 190.5, where the granite was found to be essentially sound. The fault was located between el 198.4 and 200.0. Hole 1F-207 explored granite from el 195.0 to 168.1. Fresh tightly jointed granite characterized the core from this hole.

53. After consideration it was determined that a large-size hole was necessary to further study the foundation rock. Shaft 4F-50, a 10 ft by 12 ft timbered hole, was located heelward of the Monolith 19 control line. The shaft was bottomed at el 184.3. From this point a short 6 ft by 7 ft drift was excavated landward and parallel to the control line. The base level was maintained 2-3 ft below the fault zone in order to expose a typical cross-section of the fault material. The exposed granite was similar to that cored by hole 1F-128 and discussed in preceding paragraphs. The fault material was meteoric quartz, fault gouge, and decomposed granite. The underlying material was a fresh, brittle granite. All the rock lying above the fault footwall was subsequently excavated. Jointed tended to be short and far spaced and predominately filled with a soft, green clay. Granite underlying this foundation consists of the alteration plated fault footwall in the upstream area; whereas the downstream area consists of angular shaped fresh granite.

Monolith 20

54. Monolith ranges from 199 to 220 ft long at its base, and rises 285.3 to 290.8 ft above its foundation. Half of this monolith is included in the gated overflow spillway section. The location of all drill holes and other pertinent features of this area are shown in Figure 21. None of the pre-construction holes penetrated into the present foundation therefore a discussion of said holes will be neglected. During construction, holes 1F-208 and 1F-210 were drilled and bottomed at elevations 180.7 and 171.8, respectively. Both holes encountered the fault zone, below which the granite was sound with tightly fitted joints. A third hole, 1F-208, revealed the same material present as discussed above. Only a small portion of Monolith 20 was affected by the fault zone, that area being confined mostly heelward of the

control line. The fault had the same characteristics as previously described in preceding paragraphs.

55. Joints present tended to trace less than 50 ft; a few were lightly chloritized; some were filled with meteoric quartz; others were tightly filled with chloritized or argillized material. Two of the joints that trace the entire width of the foundation were designated as minor shear zones, showing normal movement. The Monolith 20 foundation is exceedingly rough and predominately characterized by fresh to slightly weathered granite. A small microdiorite dike is present near the center of the foundation. The predominate foundation grade is el 195.

Monolith 21

56. Monolith 21 ranges from 193 to 199 ft long at its base, and rises 270.3 to 285.3 ft above its foundation. The location of all drill holes and other pertinent features of this area are shown in Figure 22. Preconstruction hole 1F-41 was put down near the heel of Monolith 21 and explored granite between el 318.4 and 179.7. Between el 205.4 and 204.9 the low angle fault was represented by sericitized chunks of soft granite. Below this elevation the granite was sound and joints appeared to be slightly filled with argillaceous material. Two holes, designated 1F-173 and 1F-209, were drilled into the foundation after some initial excavation had taken place. Hole 1F-173 was bottomed at el 213.1 and indicated sound granite, although the joints were open and coated with oxidation products. Hole 1F-209 was bottomed at el 180.4 with the entire reach showing jointed, chloritized granite.

57. A large-size hole was placed in this area to further study the material. Shaft 4F-51, a 10 ft by 12 ft timbered hole, was placed in the Monolith 21 foundation. The fault zone was encountered at el 221. A short drift, designated 4F-51B, was driven toeward from the base of shaft 4F-51. A second drift, designated 4F-51A, was driven landward from the base of shaft 4F-51. This latter drift was eventually driven along the fault to a point near the Monolith 23-24 joint line, to expose the nature of the fault zone beneath Monoliths 21, 22, and 23. The layout of this shaft and its associated drifts is shown in Figure 24. In the shaft and both drifts, the rock was found to be slightly to intensely weathered. All the rock associated

with the shaft and its drifts was subsequently removed and they were back-filled with concrete.

58. In the Monolith 21 foundation area the fault showed a noticeable decrease in thickness. The fractures associated with the fault did not show the extent of decomposition present in the previous monoliths. Joints were relatively tight either filled with argillized or chloritized products. Most showed normal movement with some showing slickensides. None of the granite beneath Monolith 21 is weathered to any appreciable extent. However, toeward much of the surface is coated with a layer of light-green, chloritized granite which grades downward into fresh granite.

Monolith 22

59. Monolith 22 ranges in base length from 178 to 185 ft and in height from 233.0 to 264.5 ft above its foundation. The location of all drill holes and other pertinent features of this area are shown in Figure 23. One pre-construction hole designated 1F-40 (battered N 54°15' E at an angle of 59°28') was drilled into the foundation material to determine conditions of weathering. The low angle fault represented by broken, altered and weathered granite was encountered between elevations 235.0 and 233.6. Below the fault, joints were essentially tight, and the granite was weathered in relatively minor degrees. Nine post-construction holes were drilled into the foundation and are discussed following. Hole 1F-127 was drilled near the control line and explored granite between el 286.1 and 184.7. The fault was discovered at el 239.1, with the granite above being slightly to intensely weathered. The granite below the fault was sound and locally weathered adjacent to open faults. Drilled from el 286.0 to 225.8, hole 1F-148 intercepted alternating zones of sound to intensely weathered granite. The fault once again was discovered at el 235.4 to 234.7, with the rock beneath being fresh and tightly jointed. Hole 1F-191 was bottomed at el 221.9 and encountered the fault at el 244.9 to 242.9. Below this the granite was fresh and tightly jointed. Holes 1F-211 through 1F-216 explored granite all of which occurred beneath the low angle fault zone. Granite from each of the holes was fresh and tightly jointed. The fault was located in each hole with the average elevation being approximately 215.

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60. Drift 4F-51A, a 6 ft by 7 ft timbered shaft, explored the granite associated with the fault through Monolith 22. This is the same drift associated with shaft 4F-51, which was began in Monolith 21. In the drift many feathering fractures, parallel to and tangential with the fault zone, offshot into granite of the hanging wall. It was decided that four more shafts, designated 4F-52 through 4F-55, should be placed in the Monoliths 22-23 area. A layout of mining operations associated with the shafts and their respective drifts is shown in Figure 24. Shafts 4F-52 and 4F-53, with their respective drifts, were founded under Monolith 22. It was determined that all rock lying above the fault zone in Monolith 22 should be removed. In the localities of shafts and drifts, excavation had removed as much as 6 ft of rock below the fault zone. All shafts and drifts lying below foundation grade were backfilled with concrete. Monolith 22 is founded on granite which lies below the low angle fault; much of the foundation surface consists of the footwall surface of the fault. The alteration plated surface of the fault comprises nearly 80 percent of the foundation surface.

61. Beneath this monolith there is a subsidiary fault which parallels the main fault. This fault consisted of a series of fractures, each discontinuous but overlapped by a continuing fracture. They had the same characteristics as previously described for the main fault. Numerous joints crossed the entire foundation width, but all were tight. None of the Monolith 22 foundation was weathered to an appreciable extent. With the exception of the three zones associated with the shafts, the entire surface consists of the alteration coated fault footwall surface. That granite associated with the shafts is fresh and quite sound.

Monolith 23

62. Monolith 23 has a base length which ranges from 142 to 144 ft, and rises to heights of 199.5 to 211.5 ft above its foundation. The location of all drill holes and other pertinent features of this monolith are shown in Figure 25. A layout of the shafts associated with this area are shown in Figure 24. None of the pre-construction holes drilled in this area reached foundation material and subsequently will not be discussed. Several holes designated: 1F-123, 1F-147, 1F-171, 1F-204, 1F-206, 1F-217, and 1F-123 were

drilled into the foundation material post-construction. Each hole encountered the fault zone, and revealed fresh tightly jointed granite lying beneath.

63. Two shafts 4F-54 and 4F-55 with their associated drifts were placed into the Monolith 23 foundation. From the results of the mining operations, it was determined that the foundation for this monolith should be founded on rock lying above the fault. The shafts and drifts were subsequently back-filled with concrete. Closely associated with the presence of the underlying fault, the weathering of the Monolith 23 foundation rock is generally well advanced. Approximately 50 percent of the foundation is composed of rock that is slightly to intensely weathered. The low angle fault zone is located some 10 to 35 ft below foundation grade. Numerous open joints were present; and along each the adjacent granite was well weathered and very soft. The land-ward half of the surface area contained several parallel joints which were bordered by as much as 0.8 ft of alteration plate. Reflecting the number and degrees of openness of joints present, the weathering in this foundation is quite developed.

Monolith 24

64. Monolith 24 is 143.5 ft long at the base and rises to a maximum height of 211.5 ft above its foundation. The location of all drill holes and other pertinent features of this area are shown in Figure 26. One hole desig-nated 1F-39 (battered N 50° E at an angle of 58°) was placed into the founda-tion beneath this monolith, pre-construction. This hole was drilled from the toe of Monolith 23 at an angle such that it terminated at el 221.5 beneath Monolith 25. The hole first entered sound rock at el 296.8 where it continued to the bottom of the hole. Several holes were placed into the foundation post-construction. Hole 1F-22 was drilled from el 311.5 to el 209.1. Below el 285.0 the granite was sound and very little core loss occurred. Hole 1F-152 was drilled to bottom el 261.5, sound relatively unweathered granite occurred below el 294.0. Core hole 1F-172 was drilled horizontally toward Monolith 26 along el 289.2, all the rock explored was slightly weathered gran-ite. Hole 1F-192 was drilled just upstream of the Monolith 25 heel and was for the purposes of locating the fault. The actual fracture which defines the fault could not be determined from the hole drilled to bottom el 236.7. Core hole 1F-200 was a secondary exploratory hole at the heel of Monolith 25

drilled to locate the fault. A minor fracture at el 266.8 was encountered and interpreted as the fault extension. Core recovery from hole 1F-202, located near the toe of Monolith 25, was 100 percent. All the core represented fresh to slightly weathered granite. Two thinly filled fractures at elevations 267.0 and 262.0 were interpreted to be fault extension fractures.

65. From the study of bore hole photographs in several of the holes, no alarming weathering conditions were detected. Joints of this foundation area were commonly filled with oxidation products and nearly all were bordered by iron-stained, fine-grained, altered granite. There was a zone of rupturing, which crossed the Monoliths 24-25 joint line and was composed of moderately to intensely weathered granite. Approximately one-half of the surface granite beneath Monolith 24 is of a fresh to slightly weathered quality. With the exception of the granite surrounding the rupture zone, the remaining foundation rock is moderately to slightly weathered granite. The relief of the Monolith 24 foundation ranges from el 302 at the intersection of the toe with the Monoliths 24-25 contraction joint to el 297 at the intersection of the heel and the Monoliths 23-24 contraction joint.

Monolith 25

66. Monolith 25 is 130 ft long at its base and approximately 198 ft in height. The location of all drill holes and other pertinent features of this area are shown in Figure 27. Pre-construction hole 1F-39 was used to investigate foundation conditions of this monolith. The discussion of this hole and its results is presented in paragraph 26 above. The following holes were drilled into the foundation after some initial excavation had occurred. Hole 1F-120 was bottomed at el 291.0, where sound granite was encountered at el 296.7. Hole 1F-121 was collared at el 330.7 and bottomed at el 230.0. Zones of persistent core loss occurred as low as el 299.7 below which the rock was sound. Drilled at an angle of 30° and located heelward of the control line, hole 1F-168 was located between elevations 296.1 and 275.9. All the core from the hole was sound and represented slightly weathered to fresh granite having several open joints. Hole 1F-201 was located adjacent to the heel of Monolith 25, and was for the purpose of accurately locating and photographing the fault. At elevations 293.1 and 291.4, the low angle fault was encountered. The fracture fillings were so minute that the fault condition in the

vicinity of this hole appears to be of minor importance. Bore hole photographs indicated that this interpretation was correct. Hole 1F-203 was placed at the toe of the monolith to study the condition of the fault. The results indicated that the fault was of little importance and appears to be minor. Blasting during the abutment excavation disturbed and displaced a local zone of granite located upstream of the control line, which required some later dental excavation.

67. Heelward of the rupture zone were five joints along which there occurred as much as 0.5 ft of altered granite. These joints were open as much as 1/4 in. and traced only a few tens of feet. Only two relatively small areas of the Monolith 25 foundation granite is intensely to moderately weathered. All the other rock quality is moderately to slightly weathered; moderate weathering occurs along the joints, and the slightly weathered granite was exposed in areas free from jointing.

Monolith 26

68. Monolith 26 is 118.6 ft long at its base and rises 173 ft above its foundation. The location of all drill holes and other pertinent features of this area are shown in Figure 28. Pre-construction hole 1F-38 (battered 52° in a direction S 02° E) was drilled approximately 25 ft upstream from the heel of Monolith 26. Below el 299.9 the core represented zones of soft, weathered rock alternating between zones of sound granite weathered to lesser degrees. Two holes were placed into the foundation after some initial excavation had occurred. Hole 1F-110 encountered sound granite between elevations 325.2 and 289.1. Hole 1F-146 was drilled to a bottom elevation of 283.7 and revealed sound foundation rock below el 317.7.

69. Six parallel joints, spaced 2 to 4 ft apart, were exposed in an area located 7 to 60 ft downstream of the control line and immediately adjacent to the Monoliths 26-27 construction joint. Each was bordered by an alteration zone and were filled with broken quartz. The majority of the remaining joints were relatively short and filled with oxidation products. However, two joints traced the entire monolith foundation and were lined with linear exfoliation ranging in width from 0.2 to 0.5 ft. An unusual zone of rupturing, which fades out near the center of the foundation, was observed tracing parallel to the control line. This zone was composed of a series of

short, steplike fractures. The greater percentage of foundation rock beneath
Monolith 26 ranges from slightly to moderately weathered. The exception being
two zones associated with the joints previously described. These areas show
granite that is moderately to intensely weathered. Joint footwalls, terminat-
ing against rough, irregularly broken blast surfaces characterize most of the
Monolith 26 foundation area.

Monolith 27

70. Monolith 27 is 118.6 ft long at its base and rises approximately
173 ft above its foundation. The location of all drill holes and other perti-
nent features of this area are shown in Figure 29. Hole 1F-37 (battered 60°
in a direction of N 60° E) was drilled into the foundation prior to any con-
struction or excavation. The hole was bottomed at el 221.2, and entered sound
granite at el 304.7. Two post-construction holes were placed into the Mono-
lith 27 foundation. Hole 1F-97 drilled from el 321.0 to 216.6, encountered
sound rock below el 318.1. Hole 1F-145 revealed that sound rock existed
beneath el 323.4.

71. Major of the structural features beneath Monolith 27 was the trace
of the low angle, minor fault which was extensively exposed beneath monoliths
of the channel section of the Concrete Gravity Dam. It had the same charac-
teristics as described in previous paragraphs. The northwesterly half of the
foundation surface was broken by several discontinuities and overlapping
joints. Several parallel joints crossed the foundation downstream of the con-
trol line. Linear exfoliation was present along the majority of these joints
in a thickness of as much as 0.7 ft. Intensely to moderately weathered gran-
ite occurs throughout an area bounded on the northwest by the trace of the
minor fault and on the southeast by a line approximately following the Mono-
liths 27-28 construction joint. Here the weathering is closely associated
with the fault and adjacent joints. Intensely to moderately weathered granite
occurs elsewhere in two elliptically shaped pockets. The remaining foundation
rock beneath Monolith 27 (approximately 50 percent) is of slightly to moder-
ately weathered quality, and joints in this rock were apparently tigh.

Monolith 28

72. Monolith 28 is 111.2 ft long at its base and rises approximately 167 ft above its foundation. The location of all drill holes and other pertinent features of this area are shown in Figure 30. Four holes were placed into the foundation after some initial excavation had occurred. Hole 1F-98A was bottomed at el 214.5, and encountered sound rock at el 310.0. Hole 1F-108 was collared at el 319.6 and drilled to el 279.1. Core loss occurred as deeply as el 295.6, with this elevation being considered the top of sound rock. Hole 1F-132 reached to bottom el 232.4, where alternating zones of sound and intensely weathered granite were found to occur as low as el 244.4. Persistent areas of core loss, however, exist only as low as el 271. Hole 1F-133 was bottomed at el 258.0 and revealed the same characteristics as hole 1F-132.

73. Most prominent of the structural features beneath Monolith 28 were two joints tracing diagonally across the foundation; in addition another such joint was located heelward of the control line at the juncture of Monoliths 27 and 28. The joints contained broken quartz and were bounded by a soft zone of iron stained alteration. Practically all the rock to the southeast of the control line is intensely weathered granite. Here foundation rock occurs which is in the most advanced weathering stage of any rock beneath the Concrete Gravity Dam. Northwest of the intensely weathered area, the condition of weathering for the most part ranges from slight to moderate. Three small areas of relatively unweathered granite are shown Figure 30.

PART III: CONSTRUCTION OF GROUT CURTAIN

74. Construction of the grout curtain beneath the Concrete Gravity Dam (Albritton 1984, Hefington 1956) was undertaken to prevent excessive seepage of reservoir water beneath the dam with the consequent development of dangerous uplift pressures against its base. In consideration of the materials filling joints of the foundation, it was agreed by all that grouting should be accomplished prior to any storage in the reservoir. Grouting could not take place beneath monoliths until there was a minimum of 100 ft of concrete in place either in the monolith directly above or 100 ft either side. This limitation was extended to 125 ft of concrete for second and third zone grouting.

75. The grouting program was performed in four zones with each zone consisting of primary, secondary and tertiary holes. Main Dam specifications provided that leader pipes, for correctly and accurately aligning grout and drain holes would be inserted through the concrete interval beneath the grouting gallery flow. Zone I was drilled to a depth of 50 ft below foundation rock, Zone II 100 ft below foundation rock and Zone III 150 ft below foundation rock. Beneath Monoliths 1-13, in the Zone I reach, the holes were grouted utilizing the progressive treatment. In this procedure holes are spaced 5 ft apart, pressure-washed and grouted in continuous operations. Beneath Monoliths 14-28, in the Zone I reach, the holes were grouted utilizing the split spacing method of hole treatment. In this procedure the primary holes are first drilled (at 10 ft spacing), then secondary holes are drilled halfway between these holes and continues in like manner for tertiary and quaternary holes. Pressure washing and grouting of holes in Zones II and III, for the entire length of the Concrete Gravity Dam, were performed in the conventional split spacing manner. In drilling and grouting the primary, secondary, tertiary, and quaternary holes in each zone, the stage method was employed. In this method a hole is drilled to the bottom of the zone, grouted until refusal is encountered, then backwashed to remove grout from the hole before it can harden. In this way, the hole can then be used to later drill into the next zone. For grouting of all holes through the lower zones, two through four, packer hookups were employed for the purpose of confining the grout to the reach of the hole below. The packer would be seated at the top of the zone to be grouted; and grout, forced through the pipe, was confined to

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the reach of hole below the packer. When grout refusal through the packer was obtained, it was then removed and the hole was backfilled with grout.

76. In Monoliths 1, 2, 9, 10, 13, 14, and 28 it was found that large takes of grout in the tertiary holes of Zone III necessitated the placement of quaternary holes. In the Monoliths 1 and 2 Zone III area, large takes of grout required the placement of a fourth zone. Zone IV was drilled to a depth of 200 ft below the top of foundation. The foundation beneath Monolith 3 showed no excessive amounts of grout being taken. In the Monoliths 4, 5, and 6 reach care was taken to prevent damage to the lining of the diversion tunnel, this area was later regrouted when the tunnel plug was inserted. These monoliths as well as Monoliths 7 and 8 revealed no large takes of grout. Monoliths 9 through 13 took nearly one-half the entire quantity of grout placed during the grouting program. This was evidenced by the large number of joints in this area. The remaining monoliths had no significantly large takes of grout.

77. Drain holes, 3 in. in diameter, spaced 10 ft apart, angled 8° downstream from the vertical, and drilled approximately 3/4 the depth of the grout curtain were place. None of the holes were drilled less than 112.5 ft below foundation rock. The location of all grout and drain holes can be seen on the figures for each monolith.

PART IV: CONCLUSIONS

78. Most of the monoliths composing the Concrete Gravity Dam are
founded on fresh to slightly weathered granite. Some of the foundation is
composed of zones that show moderate to intensely weathered granite, however,
the majority of the foundation sits upon essentially fresh granite. Located
beneath the Concrete Gravity Dam are four faults and three major joint sets.
In areas surrounding the faults and joints, where the weathering was most
extensive, dental excavation was performed to reveal granite of quality high
enough for foundation material. Particular attention and care was given to
those areas associated with the faults to assure the presence of sound foun-
dation material. Kinematic analyses of potential slip surfaces and assessment
of design shear strengths were not described in the construction foundation
reports and are beyond the scope of this summary. A grout curtain was
constructed to prevent excessive seepage.

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Table A1

Foundation of Monolith	Rock Description	Comments
1	60 percent slightly to moderately weathered, 40 percent moderately to intensely weathered	Fault tracing entire length; numerous joints
2	70 percent slightly to moderately weathered, 30 percent moderately to intensely weathered	Numerous joints present, some indicating shear zones
3	40 percent slightly to moderately weathered, 30 percent moderately to intensely weathered, 30 percent fresh to slightly weathered	Numerous joint present
4	60 percent fresh to slightly weathered, 15 percent slightly to moderately weathered, 15 percent moderately to intensely weathered	The remaining 10 percent reveals a microdioritic dike, and a brecciated fault zone
5	60 percent slightly to moderately weathered, 20 percent moderately to intensely weathered, 10 percent fresh to slightly weathered	Remaining 10 percent composes a microdioritic dike, and the brecciated fault zone
6	70 percent fresh to slightly weathered, 10 percent moderately to intensely weathered, 10 percent slightly to moderately weathered	Remaining 10 percent composes a microdioritic dike, and the brecciated fault zone
7	80 percent fresh to slightly weathered, 10 percent moderately to intensely weathered, 5 percent slightly to moderately weathered	Small trace of microdioritic dike, numerous joints, some shear zones
8	80 percent fresh to slightly weathered, 15 percent moderately to intensely weathered, 5 percent slightly to moderately weathered	Some joints and shear zones

(Continued)

Table A1 (Continued)

Foundation of Monolith	Rock Description	Comments
9	70 percent fresh to slightly weathered, 15 percent slightly to moderately weathered, 15 percent moderately to intensely weathered	Numerous traces of joints and shear zones
10	60 percent fresh to slightly weathered, 20 percent slightly to moderately weathered, 20 percent moderately to intensely weathered	Extensive traces of joints present requiring some dental excavation
11	80 percent fresh to slightly weathered, 15 percent moderately to intensely weathered, 5 percent slightly to moderately weathered	Numerous joints with a few minor shear zones
12	90 percent fresh to slightly weathered, 5 percent slightly to moderately weathered, 5 percent moderately to intensely weathered	Presence of fault near toe, numerous joints
13	90 percent fresh to slightly weathered, 5 percent slightly to moderately weathered, 5 percent moderately to intensely weathered	Presence of fault near toe, numerous joints
14	95 percent fresh to slightly weathered, 5 percent slightly to moderately weathered	Presence of fault tracing near center of foundation, few fractures
15	60 percent fresh to slightly weathered, 35 percent alteration-plated surface of fault footwall, 5 percent moderately to intensely weathered	Presence of fault tracing near heel, some joints

(Continued)

Table A1 (Continued)

Foundation of Monolith	Rock Description	Comments
16	40 percent fresh to slightly weathered, 10 percent slightly to moderately weathered, 50 percent alteration-plated surface of fault footwall	Trace of fault at heel, few joints
17	60 percent fresh to slightly weathered, 10 percent slightly to moderately weathered, 30 percent alteration-plated surface of fault footwall	Trace of fault at heel, some joints
18	50 percent fresh to slightly weathered, 45 percent alteration-plated surface of fault footwall, 5 percent slightly to moderately weathered	Trace of fault at heel, some joints
19	50 percent fresh to slightly weathered, 50 percent alteration-plated surface of fault footwall	Trace of fault at heel, numerous joints, exploratory shaft 4F-50 and drift 4F-50A
20	90 percent fresh to slightly weathered, 5 percent alteration-plated surface of fault footwall, 3 percent slightly to moderately weathered	Small trace of microdioritic dike, trace of fault at heel, numerous joints
21	40 percent fresh to slightly weathered, 60 percent fresh to slightly weathered, capped by a chloritized layer 0.1 - 0.3 ft thick	Trace of fault across the heel and the entire length of the Monoliths 21-22 joint line, numerous joints, exploratory shaft 4F-51 with drifts 4F-51A and 4F-51B

(Continued)

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Table A1 (Concluded)

Foundation of Monolith	Rock Description	Comments
22	70 percent alteration-plated surface of fault footwall, 20 percent fresh to slightly weathered, 5 percent slightly to moderately weathered, 5 percent moderately to intensely weathered	Trace of fault completely borders the foundation, numerous joints, exploratory shafts 4F-53, and 4F-52 with drift 4F-51A
23	60 percent fresh to slightly weathered, 30 percent slightly to moderately weathered, 10 percent moderately to intensely weathered	Trace of fault along Monoliths 22-23 joint line, numerous joints
24	60 percent fresh to slightly weathered, 40 percent slightly to moderately weathered	Numerous joints, small rupture zone
25	90 percent slightly to moderately weathered, 5 percent fresh to slightly weathered, 5 percent moderately to intensely weathered	Trace of rupture zone with subsequent dental excavation, few joints
26	90 percent slightly to moderately weathered, 10 percent moderately to intensely weathered	Small trace of rupture zone, some joints
27	50 percent slightly to moderately weathered, 50 percent moderately to intensely weathered	Fault racing entire length of foundation, some joints
28	50 percent slightly to moderately weathered, 40 percent moderately to intensely weathered, 10 percent fresh to slightly weathered	Some joints

Table A2

Summary of Orientations for Joint Sets Faults, and Shear Zones

in Rock Below Each Monolith

| Monolith No. | Joints | | Faults and Shear Zones |
	Strike	Dip	
1	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 80° SE-SW 73° to 87° E-NE	1 fault striking N 45° E, dipping 40° to 62° NW, 0.2' to 0.8' thick w/soft clay and decom- posed granite
2	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 80° SE-SW 73° to 87° E-NE	None noted
3	N 25° E to N 60° E N 30° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 80° SE-SW 73° to 87° E-NE	Minor shear zones associated with N ° E to S 80° E joints
4	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	2 parallel faults, striking N 45° E, dipp- ing 30° to 40° NW, 0.6' to 4.0' thick w/breccia and clay
5	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	2 parallel faults, striking N 45° E, dipp- ing 30° to 45° NW, 0.2' to 4.0' thick w/breccia and clay; also 1 nearly horz. fault, 0.1' thick w/breccia
6	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault striking N 45° E, dipping 44° NW, 0.2' to 3.0' thick w/breccia; also minor shear zones assoc. w/N 80° E to S 80° E joints
7	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	Minor shear zones assoc. with N 80° E to S 80° E joints

(Continued)

Monolith No.	Joints		Faults and Shear Zones
	Strike	Dip	
8	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	36" calyx hole indicates 4 minor faults containing clay and gouge; strike direction not known for 3 of the faults; 1 fault striking E, dipping 70° W. Also 2 shear zones assoc. with N 80° E to S 80° E joints
9	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	Minor shear zones assoc. w/N 80° E to S 80° E joints
10	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	Minor shear zones assoc. w/N 80° E to S 80° E joints
11	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	Minor shear zones assoc. w/N 80° E to S 80° E joints. Some filled w/0.2' of clay and gouge
12	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	A few of the N 25° E to N 60° E joints slickensided and contain gouge; shear zones assoc. w/N 80° E to S 80° E joints; 1 minor fault, striking N 32° E to N 80° E, dipping 25° NW
13	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	1 minor fault, striking N 30° E, dipping 25° NW, many small low angle features radiating from fault
14	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault, striking approx. N 40° E, dipping 22° NW, contains 0.4' to 1.2' of soft chlorite and broken meteoric quartz

(Continued)

(Sheet 2 of 4)

Monolith No.	Joints		Faults and Shear Zones
	Strike	Dip	
15	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	32° to 37° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault, striking N 60° E dipping 17° NW 0.4' to 1.2' thick filled with soft clay and gouge
16	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault present, but was excavated down to the footwall
17	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	15° to 80° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault present, but was excavated down to the footwall: one small shear zone assoc. w/N 80° W to S 80° E joints
18	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	34° to 80° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault present, but was excavated down to the footwall; minor shear zone assoc. w/N 80° E to S 80° E joints filled w/0.1' to 0.2' of soft clay
19	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	34° to 80° NW 62° to 74° SE-SW 73° to 87° E-NE	Same as in Monolith 18
20	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	14° to 87° NW 62° to 74° SE-SW 73° to 87° E-NE	Same as in Monolith 18
21	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	21° to 79° NW 62° to 74° SE-SW 73° to 87° E-NE	3 faults present, but were excavated down to the footwall; minor shear zone assoc. w/N 80° E to S 80° E filled w/0.1' of clay
22	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault bordering on 3 sides of the monolith was excavated down to the footwall; 1 sub- sidiary fault striking N 30° E, dipping 16°-21°

(Continued)

| Monolith No. | Joints | | Faults and Shear Zones |
	Strike	Dip	
22	N 25° E to N 60° E N 80° E to S 80° E N0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	NW remains, this fault, 1' to 6' thick, contains fractured rock w/fractures filled fault gouge
23	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 58° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fault striking on N 15°-30° E, dipping 13° to 25° NW contains 0.1' to 1' of fat clay and decomposed granite, approx. 45 percent of the fault was mined and filled w/conc.; 1 sub-siding fault as in monolith 22 also present but not mined
24	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	30° to 60° NW 62° to 74° SE-SW 73° to 87° E-NE	None noted
25	N 25° E to N 60° E N 80° E to S 80° E N 0° W to N 55° W	25° to 60° NW 62° to 74° SE-SW 73° to 87° E-NE	1 fracture zone, striking N 80° E, dipping 10° to 30° NW, 2' to 3' in width
26	N 30° E to N 55° E N 80° E to S 80° E	25° to 58° NW 62° to 74° SE-SW	1 fracture zone, striking N 35° W, dipping 10° to 30° NE, 2' to 2.5' in width
27	N 30° E to N 55° E N 80° E to S 80° E	25° to 58° NW 62° to 74° SE-SW	1 minor fault, striking N 30° E, dipping 36° NW, contains 1' to 2' intensely weathered granite
28	N 30° E to N 55° E N 80° E to S 80° E	25° to 58° NW 62° to 74° SE-SW	Minor fracture zones assoc. w/N 80° E to S 80° E joints

(Sheet 4 of 4)

RETAINING WALL "B"

RIGHT WING DAM

MAIN CONCRETE DAM

TRASHRACKS

① ② ③ 4 — 5 — 6 — 7 — 8 — 9 — 10 — 11 — 12 — 13 — 14 — 15 — 16 — 17

TOWER

CONDUITS

PUMPING PLANT

Pipeline - North Fork Ditch

RETAINING WALL "A"

STILLING BASIN

ROAD

ACCESS

PENSTOCKS →

POWER HOUSE
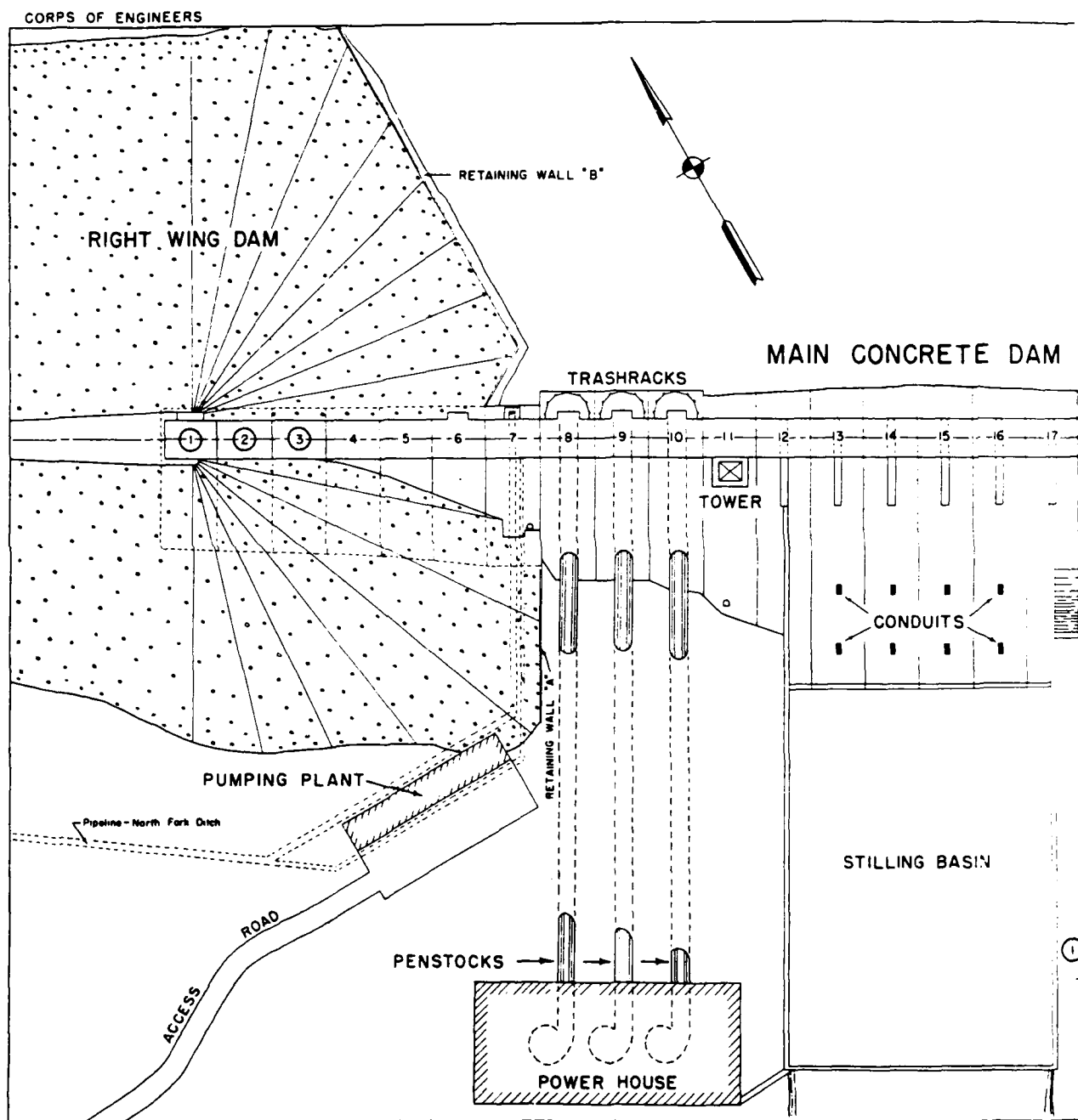
Figure 1. Plan View of Concrete Gravity Dam and Ap

MAIN CONCRETE DAM

LEFT WING DAM

ACKS

—10— —11— —12— —13— —14— —15— —16— —17— —18— —19— —20— —21— —22— —23— —24— —25— —26— —27— —28— CONTROL AXIS
S 60° 08' 08" E

TOWER

FLIP
BUCKET

CONDUITS

RETAINING WALL "E"

STILLING BASIN

Pipeline – Natoma Ditch

① ② ③ Monoliths included in this report.

-28- Monoliths previously reported.

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
PLAN OF DAM & APPURTENANCES

Submitted   Approved

SCALE

Project Office   March 1954   FR-D-9

USE

/lew of Concrete Gravity Dam and Appurtenances.

J
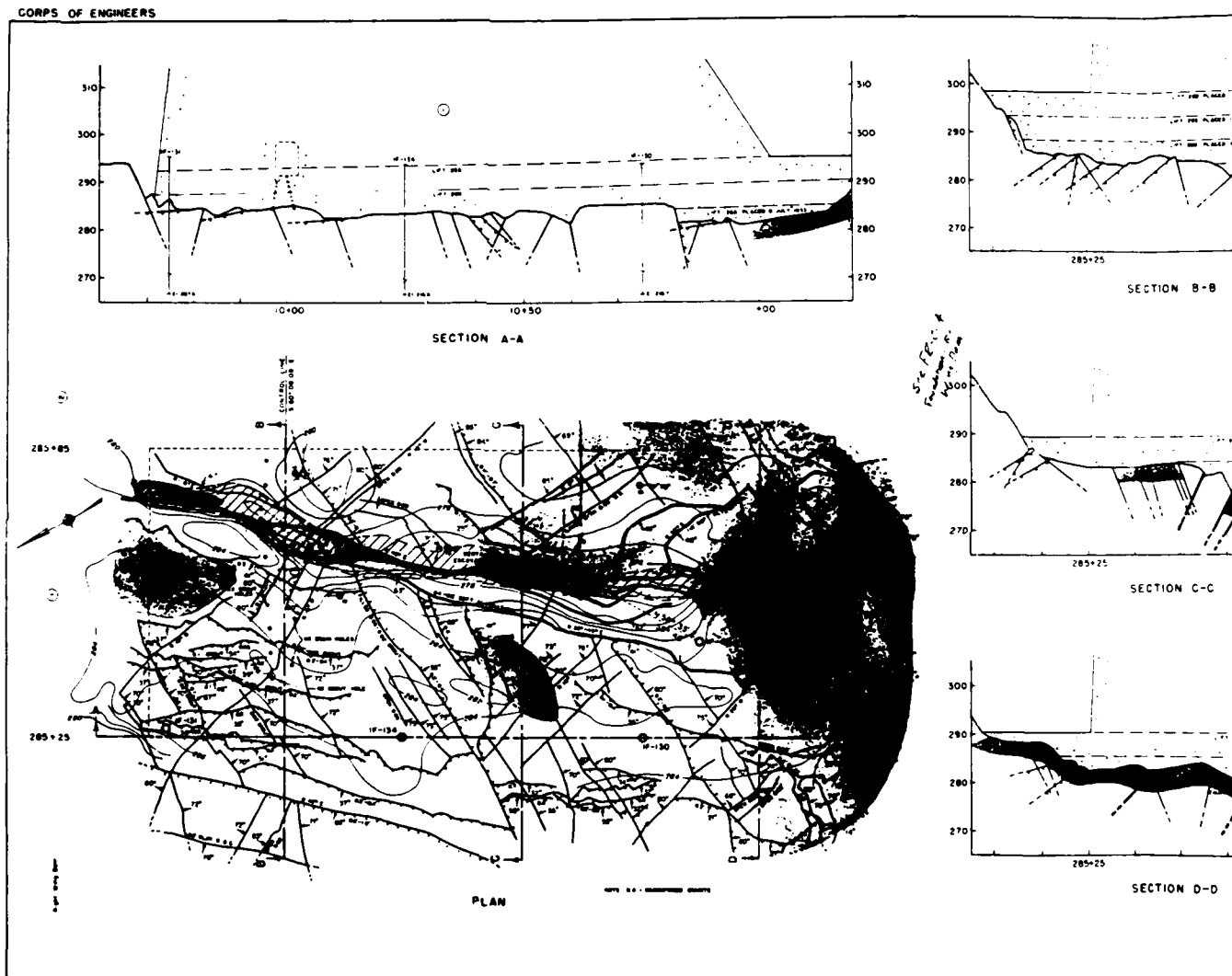
Figure 2. Plan and Sections of Concrete Gravity Dam Monolith 1.
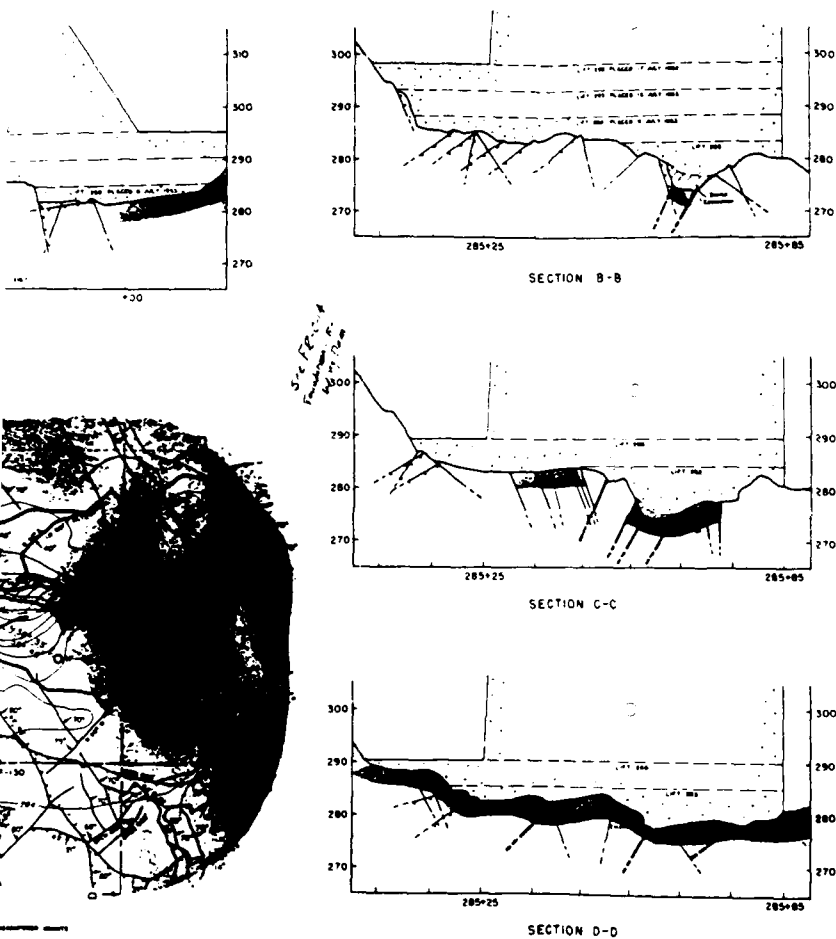
SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

Trace and dip of joints & fault

Tight joint

Fault

Altered joints striking N 40°-55° E. Small figures indicate alteration extent

Foliation-lined joints-generally strike N 75°-90° E. Small figures indicate extent of their alteration

NX cored drill hole, 2⅛" core, vertical

NX cored drill hole, 2⅛" core, angle

Location of photograph No 18

NOTE   See FR-D-Logs 24 thru 27 for exploration logs.

SCALE
CONTOUR INTERVAL 2 FEET

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH I
PLAN AND SECTIONS

PROJECT OFFICE   March 1954   FR-D-10

ions of Concrete Gravity Dam Monolith 1.
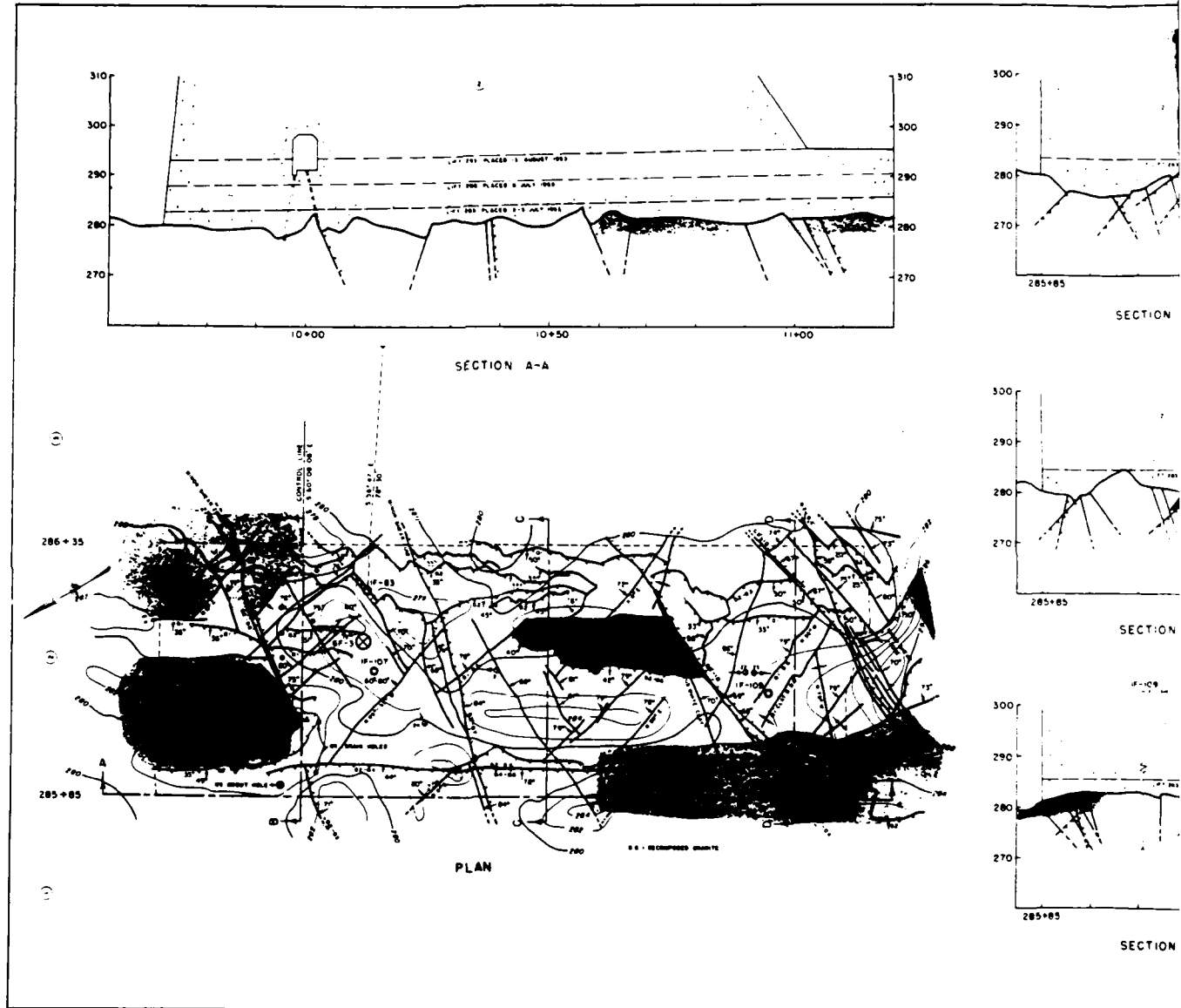
Figure 3. Plan and Sections of Concrete Gravity Dam Monolith
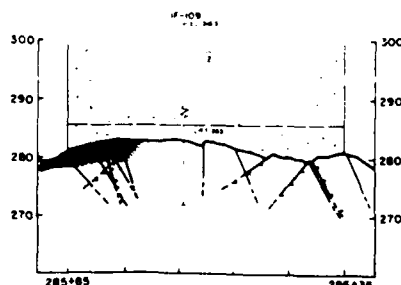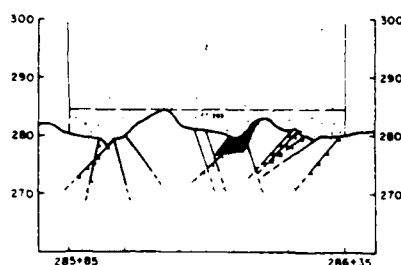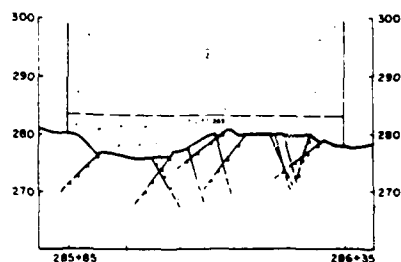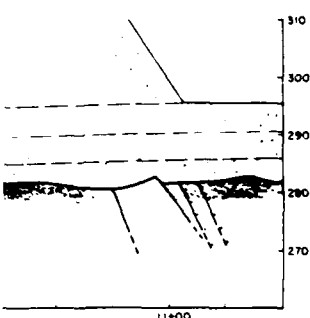
SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

Trace and dip of joints & shear zones.

Tight joint

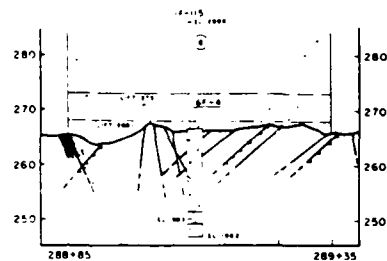Altered joints striking N 40°-55° E. Small figures indicate alteration extent

Foliation-lined joints - generally strike N 75°-90° E. Small figures indicate extent of linear alteration.

NX cored drill hole, 2⅛" core, vertical

NX cored drill hole, 2⅛" core, vertical

Calyx hole, 36" diameter

Location of photograph No. 22

NOTE See FR-D-Logs 28 thru 31 for exploration logs.
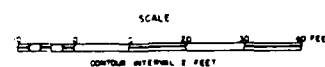
SCALE

CONTOUR INTERVAL 2 FEET

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 2
PLAN AND SECTIONS

Submitted   Approved

PROJECT OFFICE   March 1954   FR-D-11

id Sections of Concrete Gravity Dam Monolith 2.
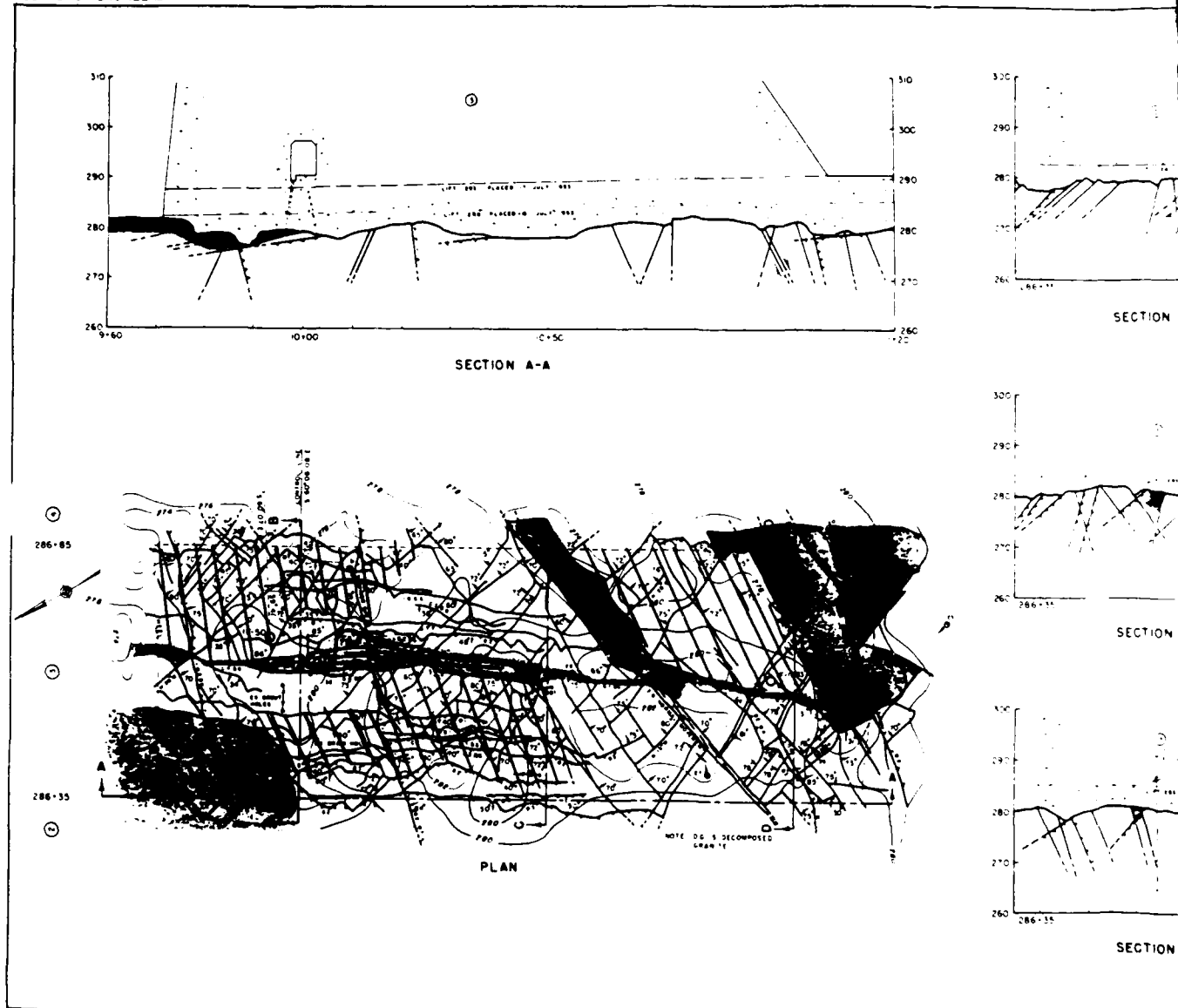
Figure 4. Plan and Sections of Concrete Gravity Dam Monol
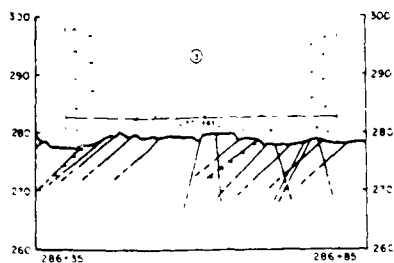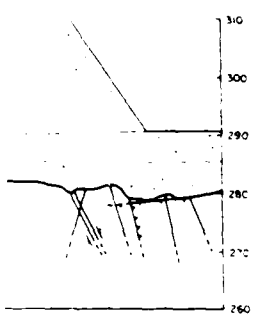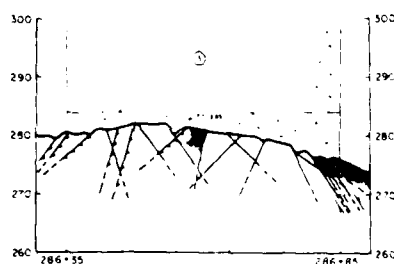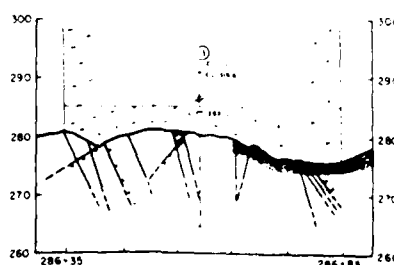
SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite) Fresh to slightly weathered

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

Trace and dip of joints and shear zones

Tight joint

Altered joints striking N 40-55 E. Small figures indicate alteration extent

Foliation-lined joints striking N 75-90 E. Small figures indicate extent of linear alteration

NX cored drill hole 2; core, vertical

NX cored drill hole 2; core, angle

Location of photograph No 27

NOTE   See FR-D Log 32 thru 33 for exploration logs

SCALE

CONTOUR INTERVAL 2 FEET
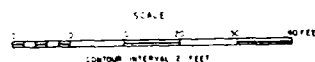
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AMERICAN RIVER, CALIFORNIA

FOUNDATION REPORT

MAIN CONCRETE DAM
MONOLITH 3
PLAN AND SECTIONS

Submitted

Approved

PROJECT OFFICE   MARCH 1954   FR-D-12

Sections of Concrete Gravity Dam Monolith 3.

1
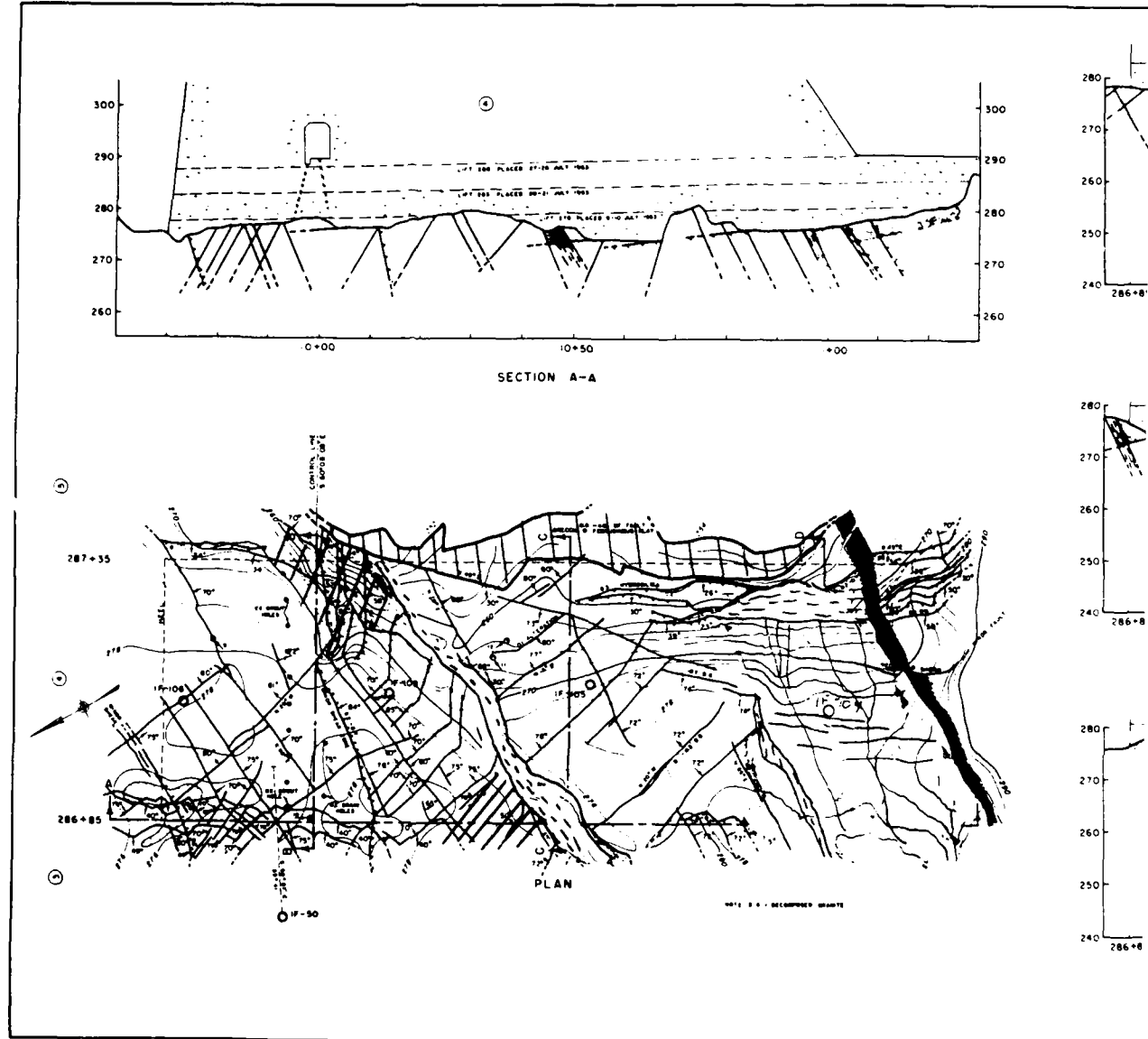
SECTION A-A

PLAN

Figure 5. Plan and Sections of Concrete Gravity Dam

SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

Brecciated fault zone

GRANITE (Quartz Diorite) Fresh to slightly weathered

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

MICRODIORITE Dike

Trace and dip of joints, faults, and shear zones

Fault

Tight joints

Altered joints striking N 40°-55° E. Small figures indicate alteration extent

Foliation-lined joints striking N 75°-90° E. Small figures indicate extent of linear deformation

IF-105 NX cored drill hole, 2⅛" core, vertical

IF-50 NX cored drill hole, 2⅛" core, angle

Location of photograph No. 9

NOTE See FR-D-Log 32 and FR-D-Logs 34 thru 37 for exploration logs.

SCALE

CONTOUR INTERVAL 2 FEET

FOLSOM PROJECT
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**MAIN CONCRETE DAM
MONOLITH 4
PLAN AND SECTIONS**

Submitted   Approved

PROJECT OFFICE   APRIL 1954   FR-D-14
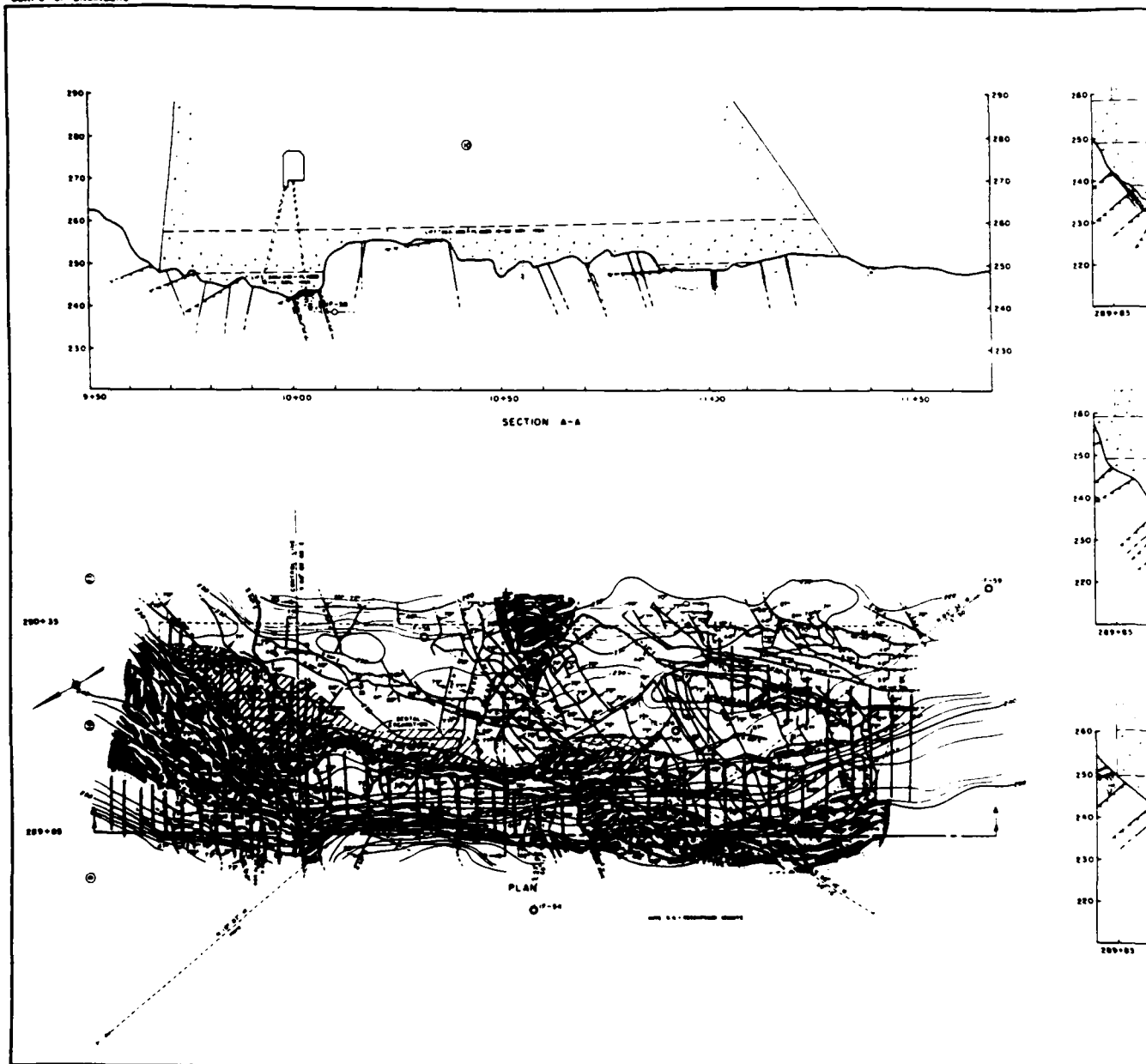
and Sections of Concrete Gravity Dam Monolith 4.

SECTION A-A

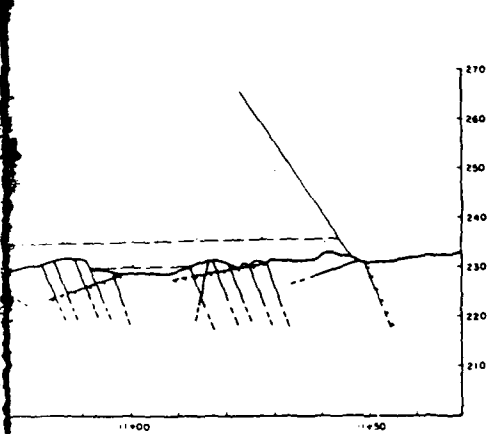PLAN

Figure 6. Plan and Sections of Concrete Gravity Dam Mon

SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

Brecciated fault zones

GRANITE (Quartz Diorite) Fresh to slightly weathered

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

MICRODIORITE Dike

Trace and dip of joints, faults, and shear zones

Fault

Tight joints

Altered joints striking N 40°-55° E. Small figures indicate alteration extent

Foliation-lined joints striking N 75°-90° E. Small figures indicate extent of linear exfoliation

NX cored drill hole 2⅛" core, vertical

EX cored drill hole ⅞" core, vertical

Timbered shaft

Location of photograph No 27

NOTE   See FR-D-Logs 38 thru 42 for exploration logs

SCALE

CONTOUR INTERVAL 2 FEET

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MAIN CONCRETE DAM
MONOLITH 5
PLAN AND SECTIONS

PROJECT OFFICE   APRIL 1954   FR-D-15

d Sections of Concrete Gravity Dam Monolith 5.

SECTION A-A

PLAN

Figure 7. Plan and Sections of Concrete Gravity Dam Mon...
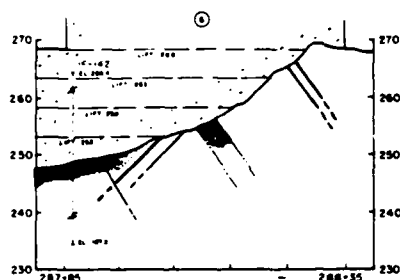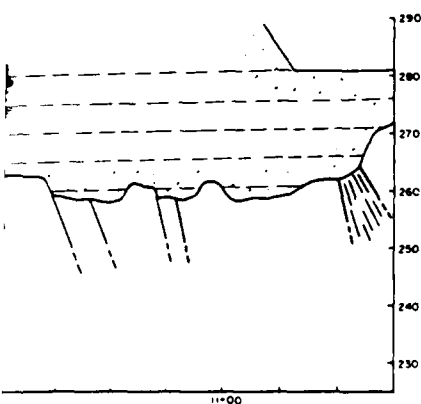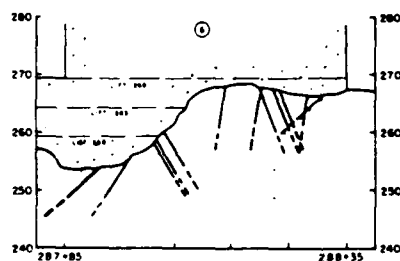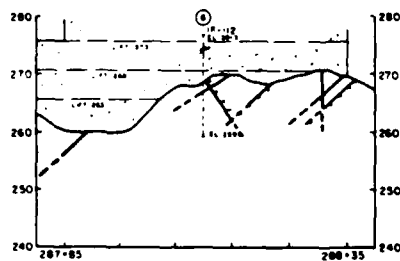
SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

Breccieted fault zone

GRANITE (Quartz Diorite) Fresh to slightly weathered

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

MICRODIORITE Dike

Trace and dip of joints faults and shear zones

Fault

Tight joints

Altered joints striking N 40°-50° E Small figures indicate alteration extent

Felsation-lined joints striking N 75°-90° E Small figures indicate extent of linear alteration

NX cored drill hole 2⅛ core vertical

NX cored drill hole 2⅛ core angle

Location of photograph No. 31

NOTE See FR-D-Log 43 thru 49 for exploration logs

DG = DECOMPOSED GRANITE
SLCK = SLICKENSIDED

SCALE

CONTOUR INTERVAL 2 FEET
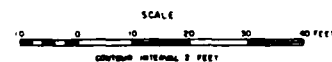
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AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 6
PLAN AND SECTIONS

Submitted   Approved
PROJECT OFFICE   APRIL 1954   FR-D-16

Plan and Sections of Concrete Gravity Dam Monolith 6.
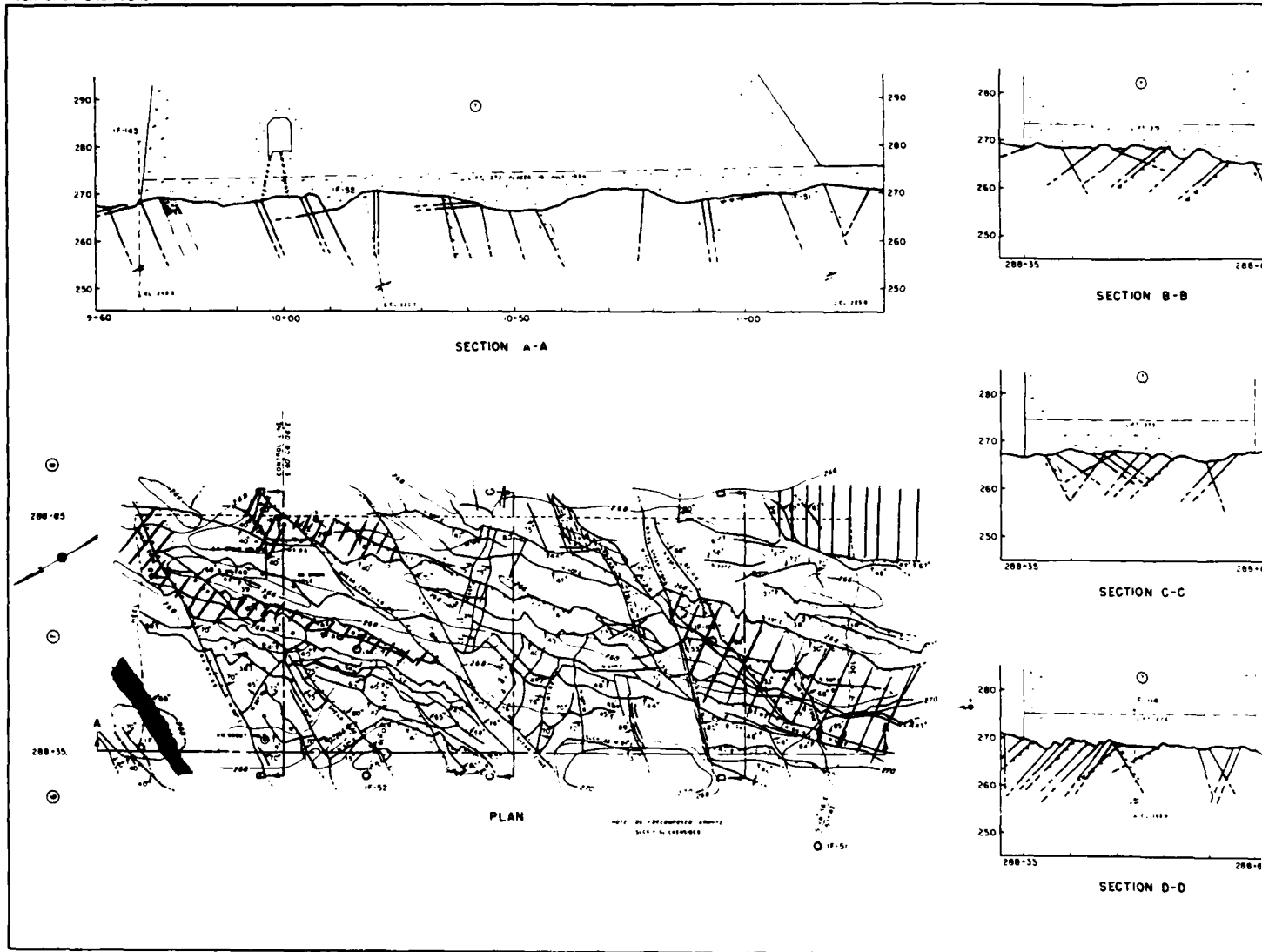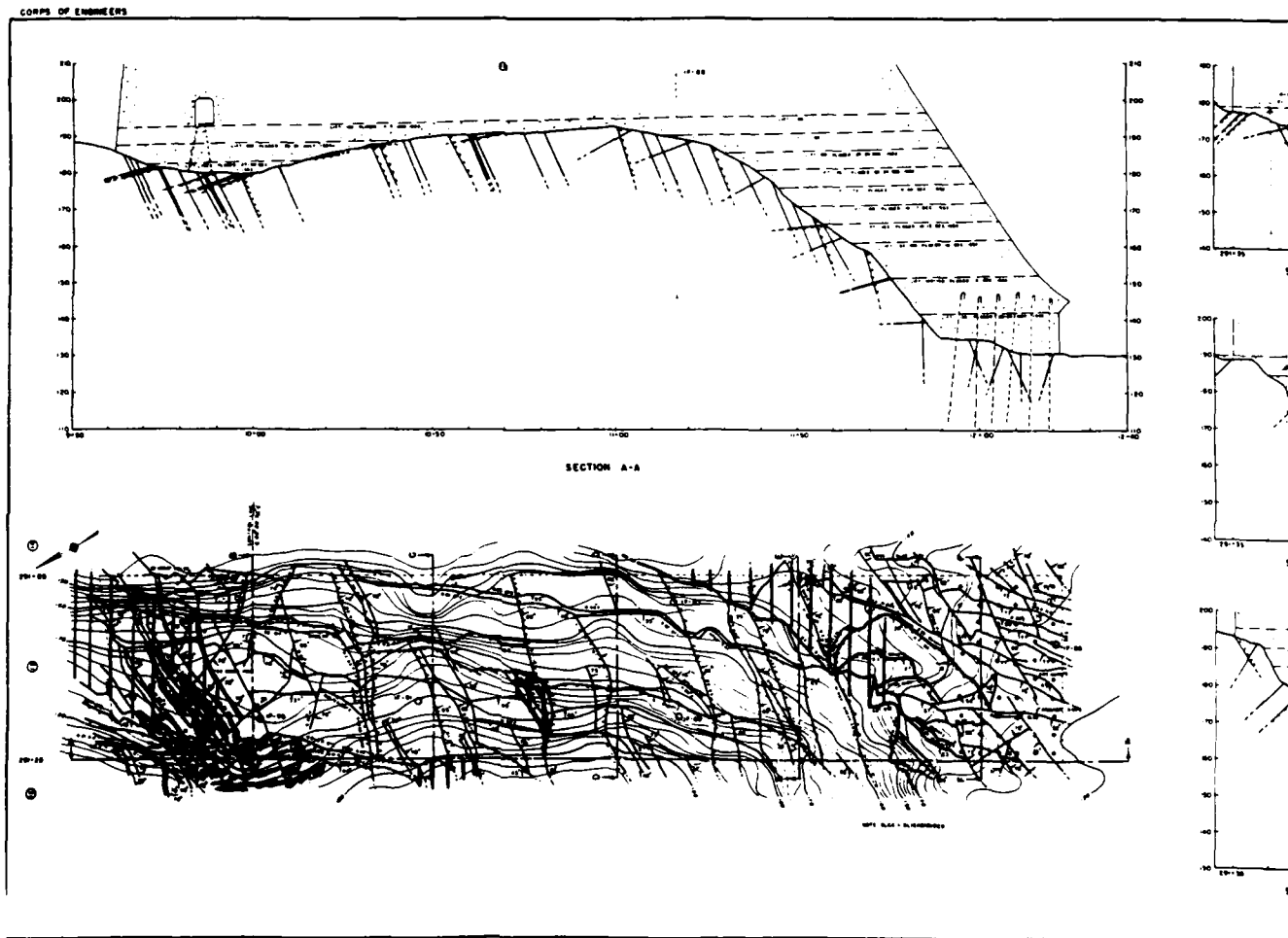
Figure 8. Plan and Sections of Concrete Gravity Dam Monolith 7.
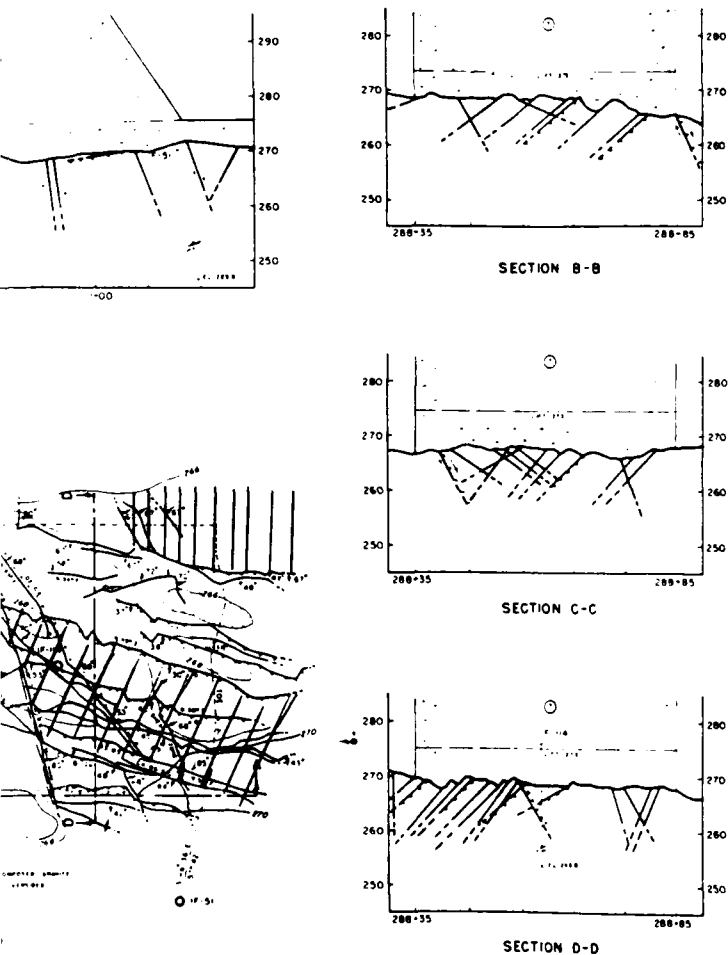
SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite) Fresh to slightly weathered

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

MICRODIORITE Dike

Trace and dip of joints and shear zones

Tight joint

Altered joints striking N 40°-55'E Small figures indicate alteration extent

Foliation-lined joints striking N 75°-90°E Small figures indicate extent of linear extension

NX cored drill hole 2⅛ core, vertical

NX cored drill hole 2⅛ core, angle

Location of photograph No 10

NOTE   See FR-D-Log 43,44 and 50 thru 52 for exploration logs

SCALE

CONTOUR INTERVAL 2 FEET

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AMERICAN RIVER, CALIFORNIA

FOUNDATION REPORT

MAIN CONCRETE DAM
MONOLITH 7
PLAN AND SECTIONS

Submitted

Approved

PROJECT OFFICE   MAY 1954   FR-D-8

Sections of Concrete Gravity Dam Monolith 7.
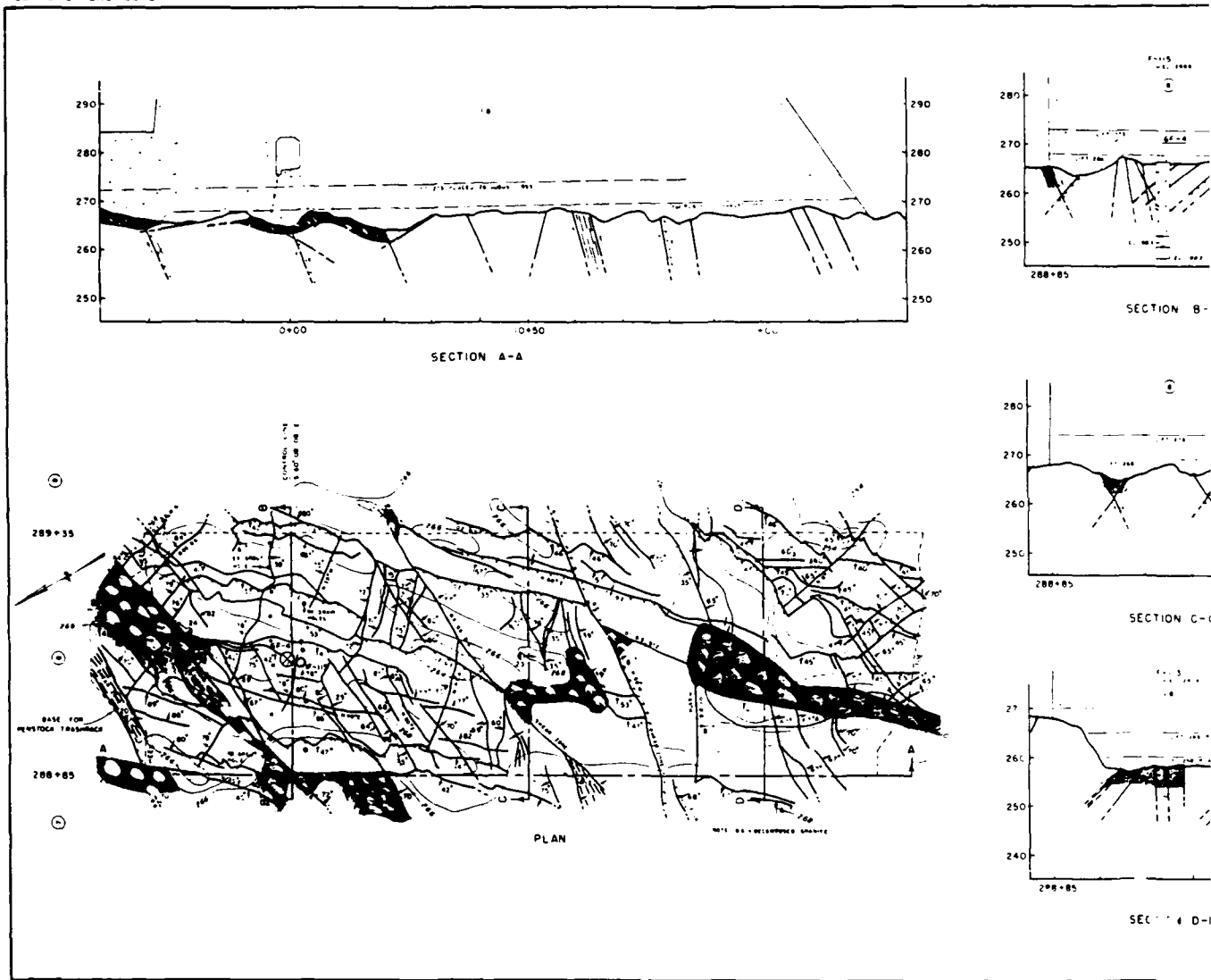
Figure 9. Plan and Sections of Concrete Gravity Dam Monolit
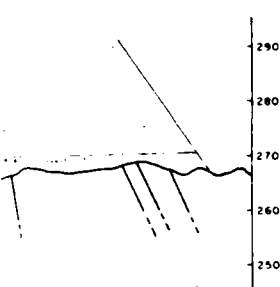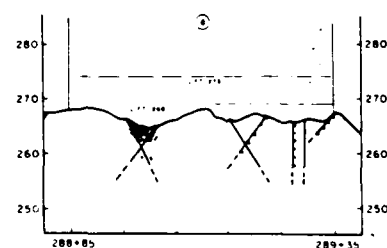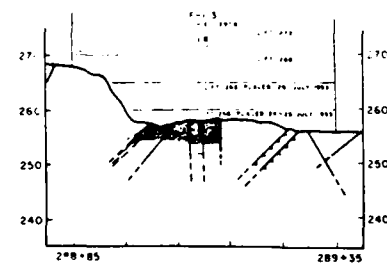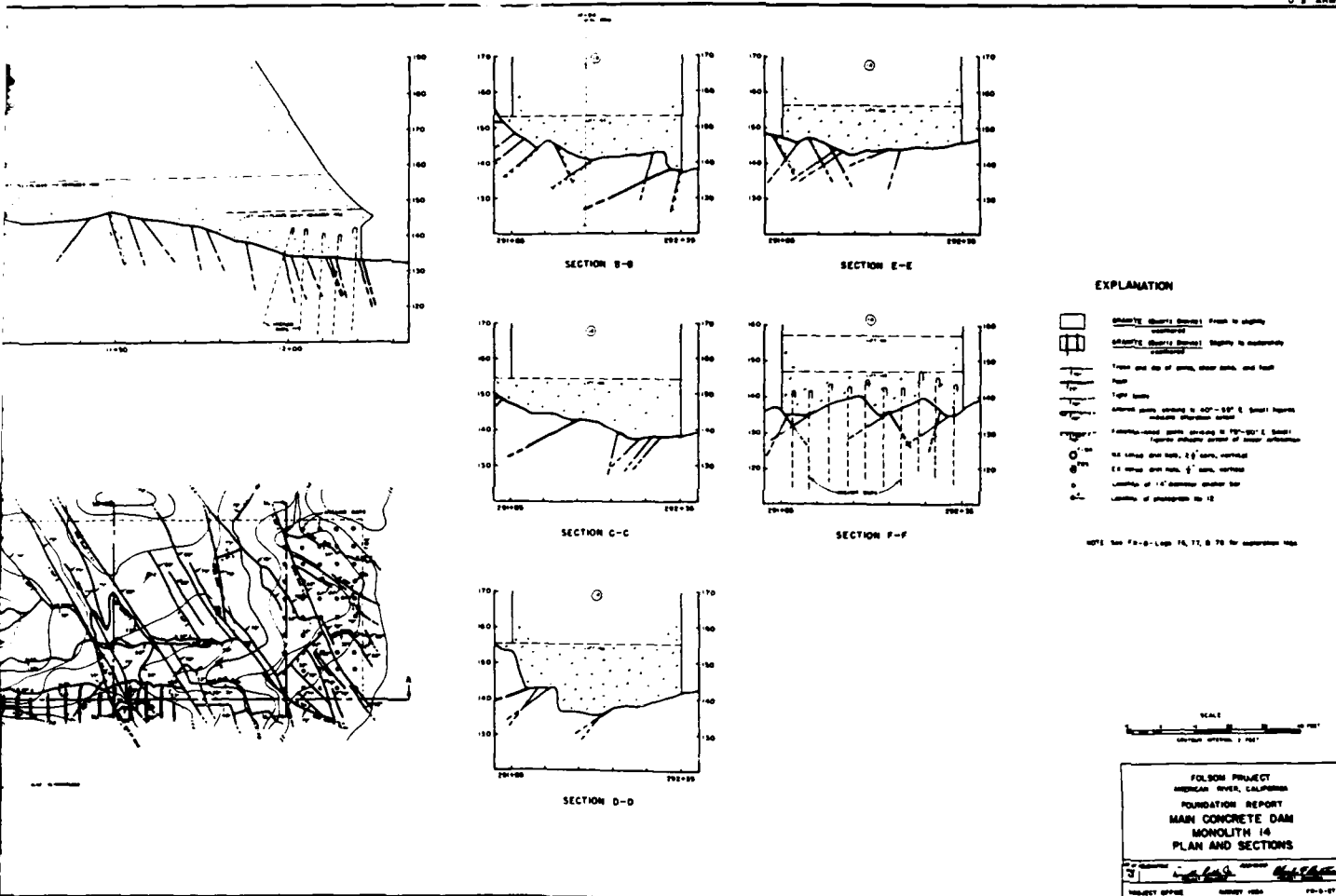
SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite). Fresh to slightly weathered

GRANITE (Quartz Diorite). Slightly to moderately weathered

GRANITE (Quartz Diorite). Moderately to intensely weathered

Trace and dip of joints and shear zones

Tight joint

Altered joints striking N 40-55 E Small figures indicate alteration extent

Foliation lined joints striking N 75-90 E Small figures indicate extent of linear exfoliation

NX cored drill hole 2; core vertical

Calyx hole 36" diameter

Location of photograph No 20

NOTE   See FR-D-Log 53 thru 55 for exploration logs

SCALE

CONTOUR INTERVAL 2 FEET

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 8
PLAN AND SECTIONS

	Submitted	Approved

PROJECT OFFICE   MAY 1954   FR-D-19

Sections of Concrete Gravity Dam Monolith 8.
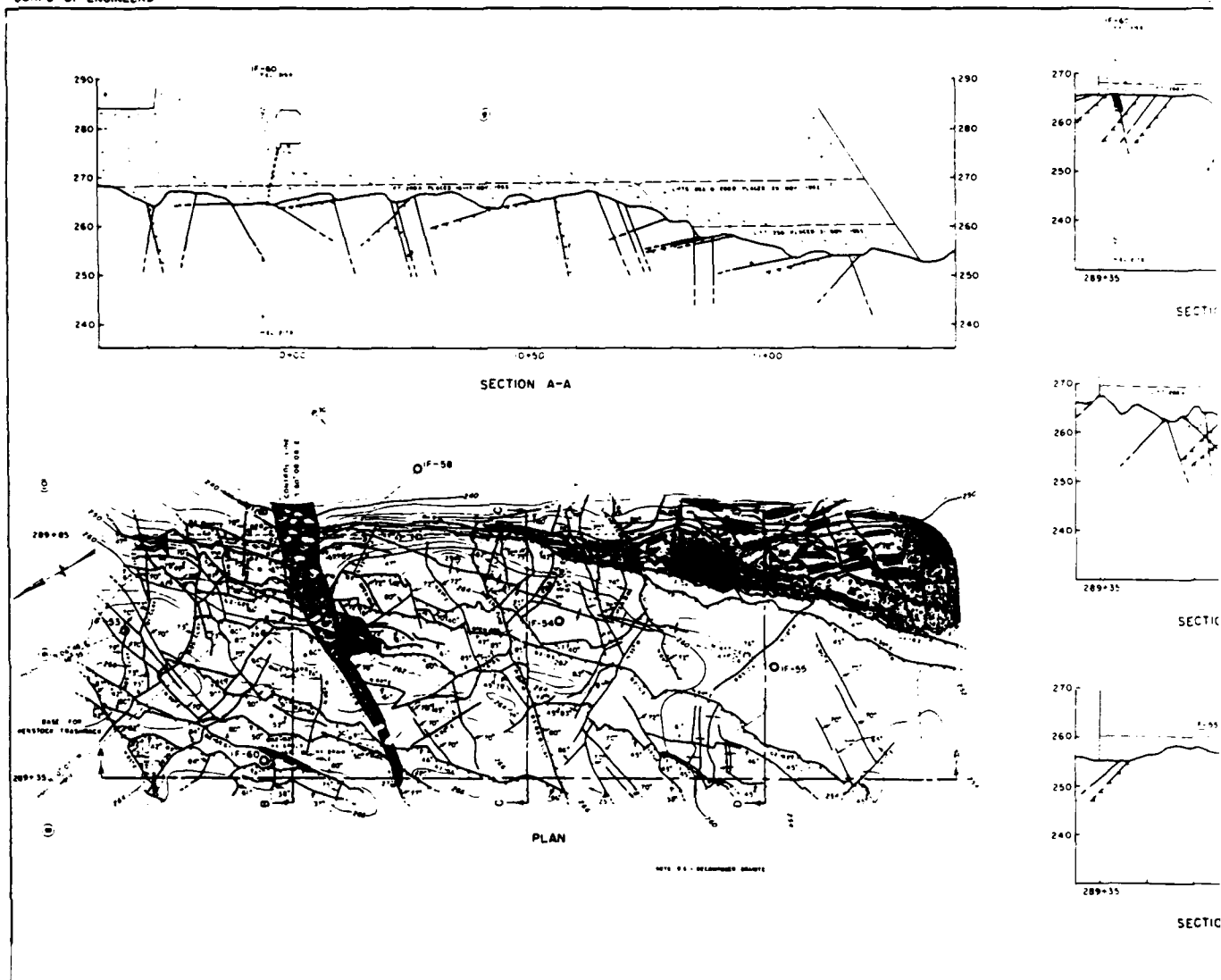
SECTION A-A

PLAN

Figure 10. Plan and Sections of Concrete Gravity Dam Monolit

SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite) Fresh to slightly
weathered

GRANITE (Quartz Diorite) Slightly to moderately
weathered

GRANITE (Quartz Diorite) Moderately to intensely
weathered

Trace and dip of joints and shear zones

Tight joint

Altered joints striking N 40°-55°E Small figures
indicate alteration extent

Foliation-lined joints striking N 75°-90°E Small
figures indicate extent of linear foliation

NX cored drill hole 2¾ core, vertical

NX cored drill hole 2¾ core, angle

Location of photograph No 28

NOTE See FR-D-Logs 56 thru 60 for exploration logs

SCALE

CONTOUR INTERVAL 2 FEET

d Sections of Concrete Gravity Dam Monolith 9.

SECTION A-A

PLAN

Figure 11. Plan and Sections of Concrete Gravity Dam Monol
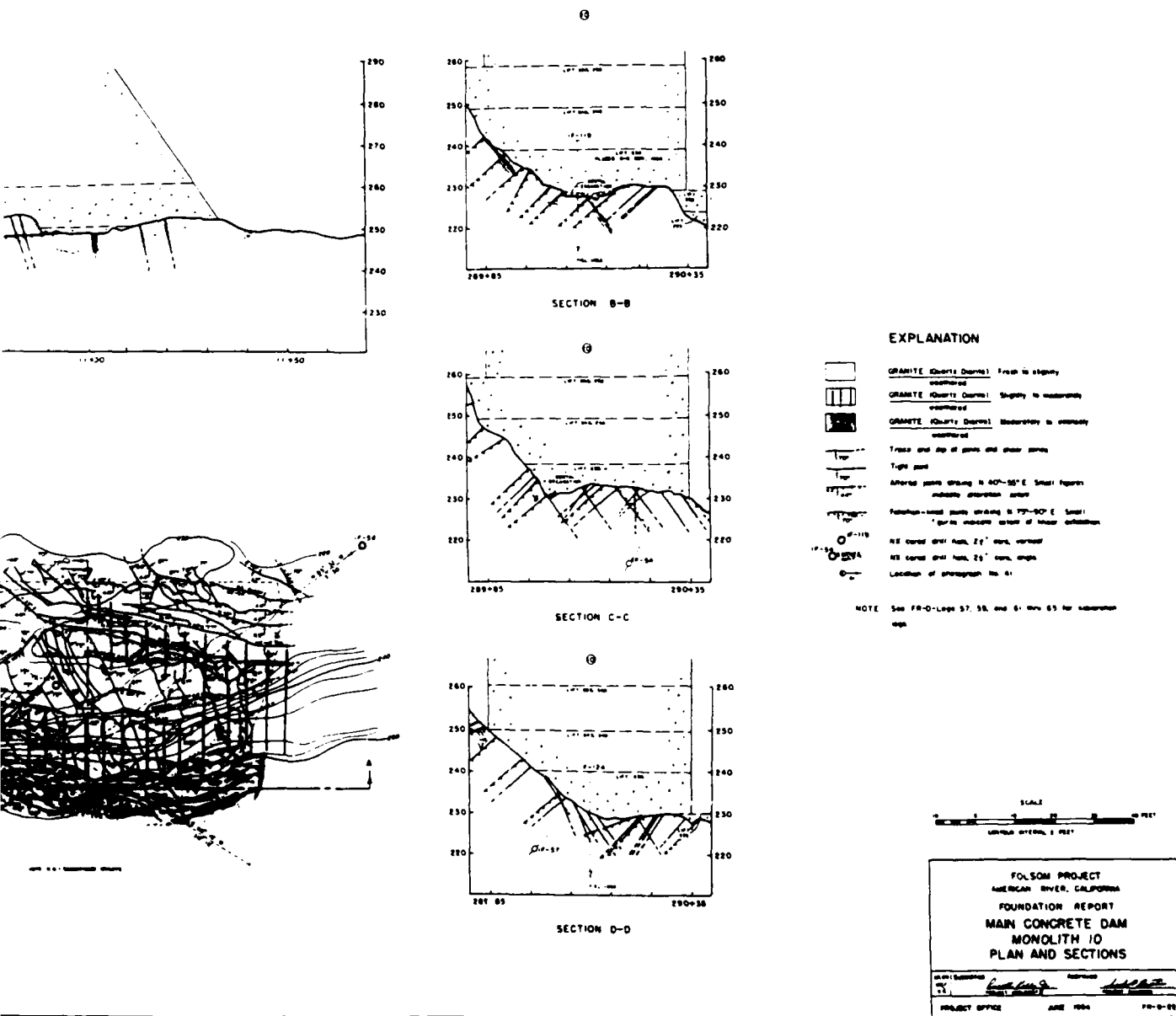
SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA

FOUNDATION REPORT

MAIN CONCRETE DAM
MONOLITH 10
PLAN AND SECTIONS

SECTION A-A

PLAN

Figure 12. Plan and Sections of Concrete Gravity Dam Mon

and Sections of Concrete Gravity Dam Monolith 11.

Figure 13. Plan and Sections of Concrete Gravity Dam M

Plan and Sections of Concrete Gravity Dam Monolith 12.

Figure 14. Plan and Sections of Concrete Gravity Dam Monoli

3d Sections of Concrete Gravity Dam Monolith 13.
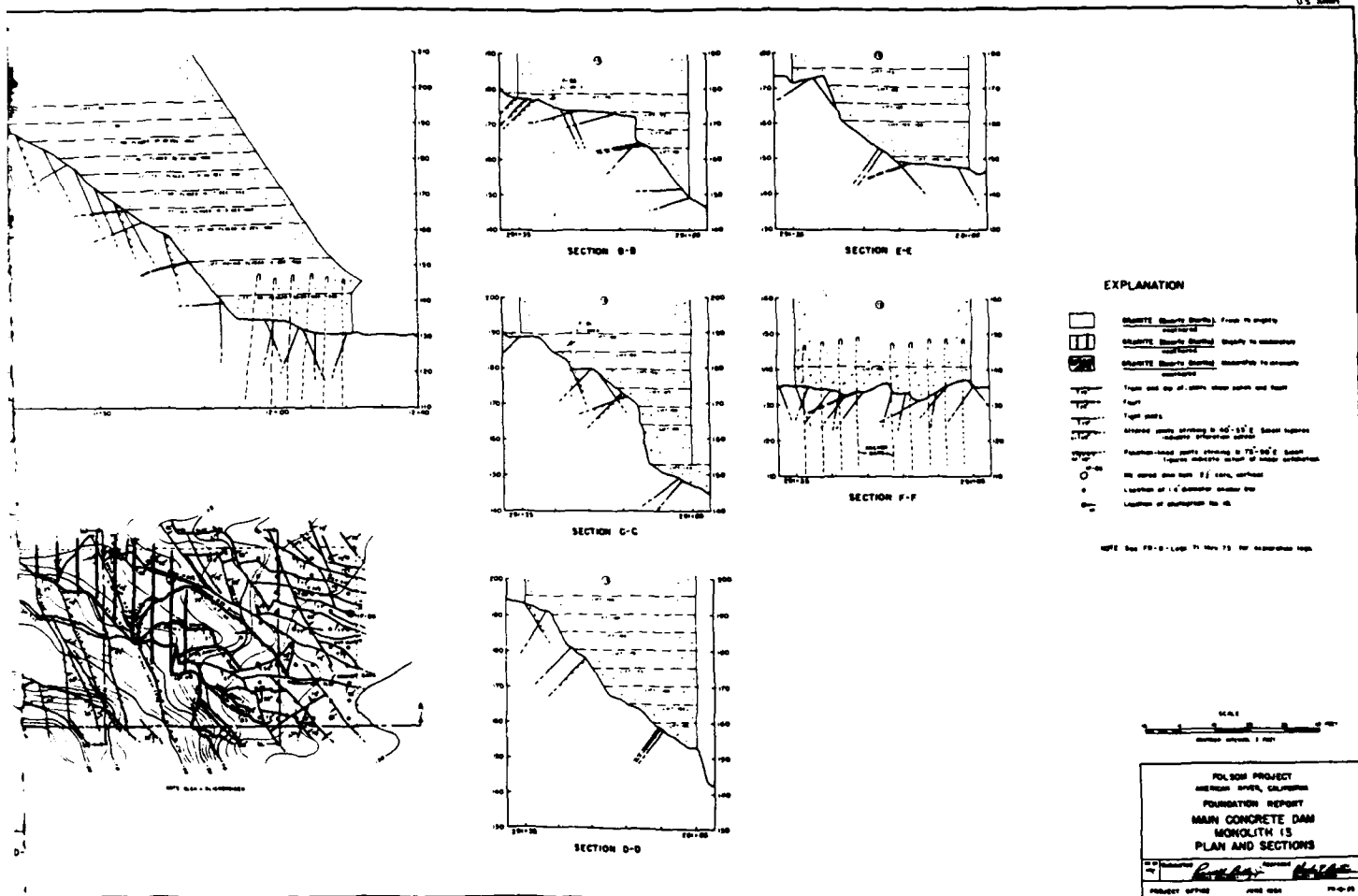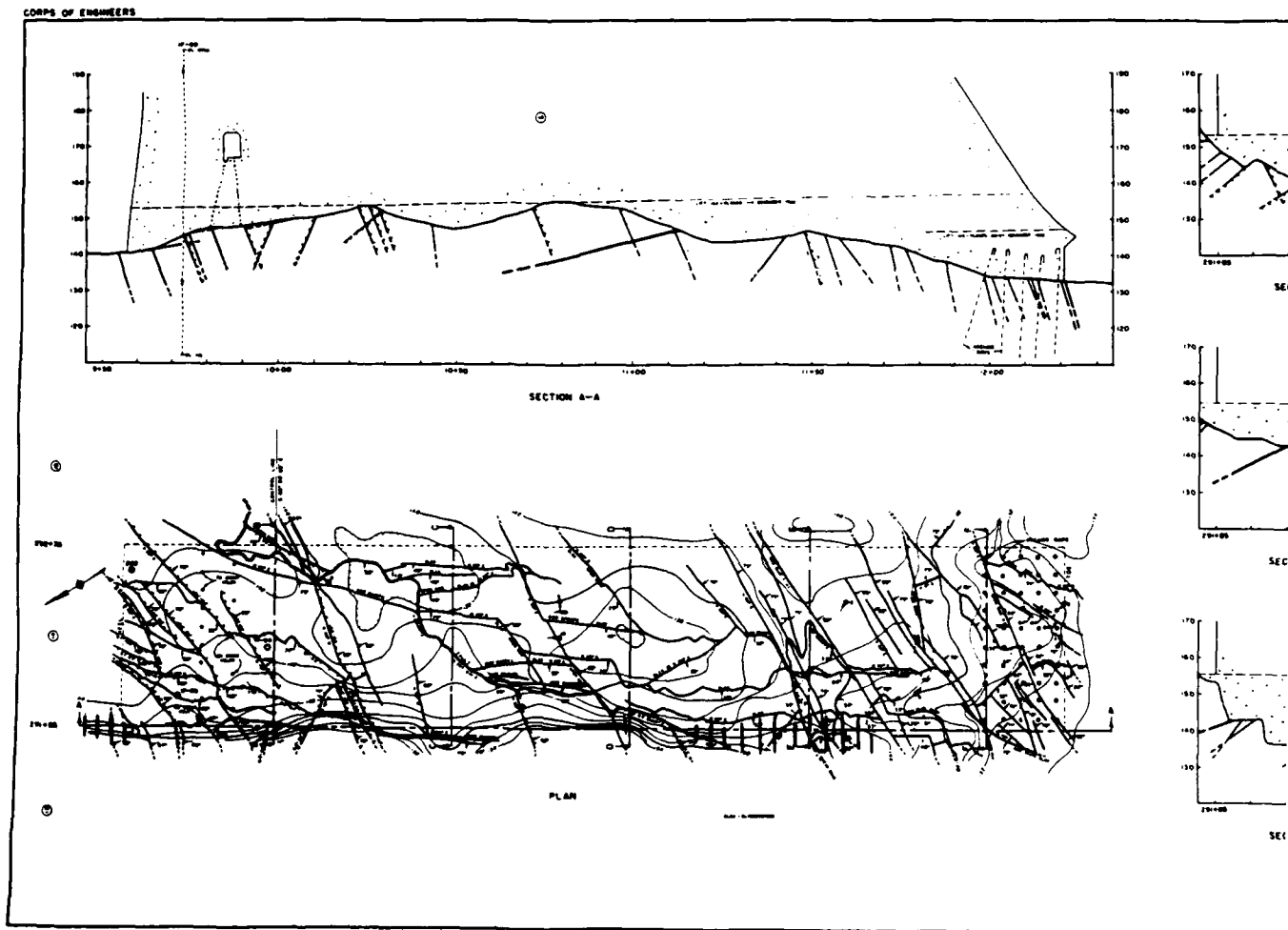
Figure 15. Plan and Sections of Concrete Gravity Dam Monoli

SECTION B-B

SECTION E-E

SECTION C-C

SECTION F-F

SECTION D-D

EXPLANATION

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 14
PLAN AND SECTIONS

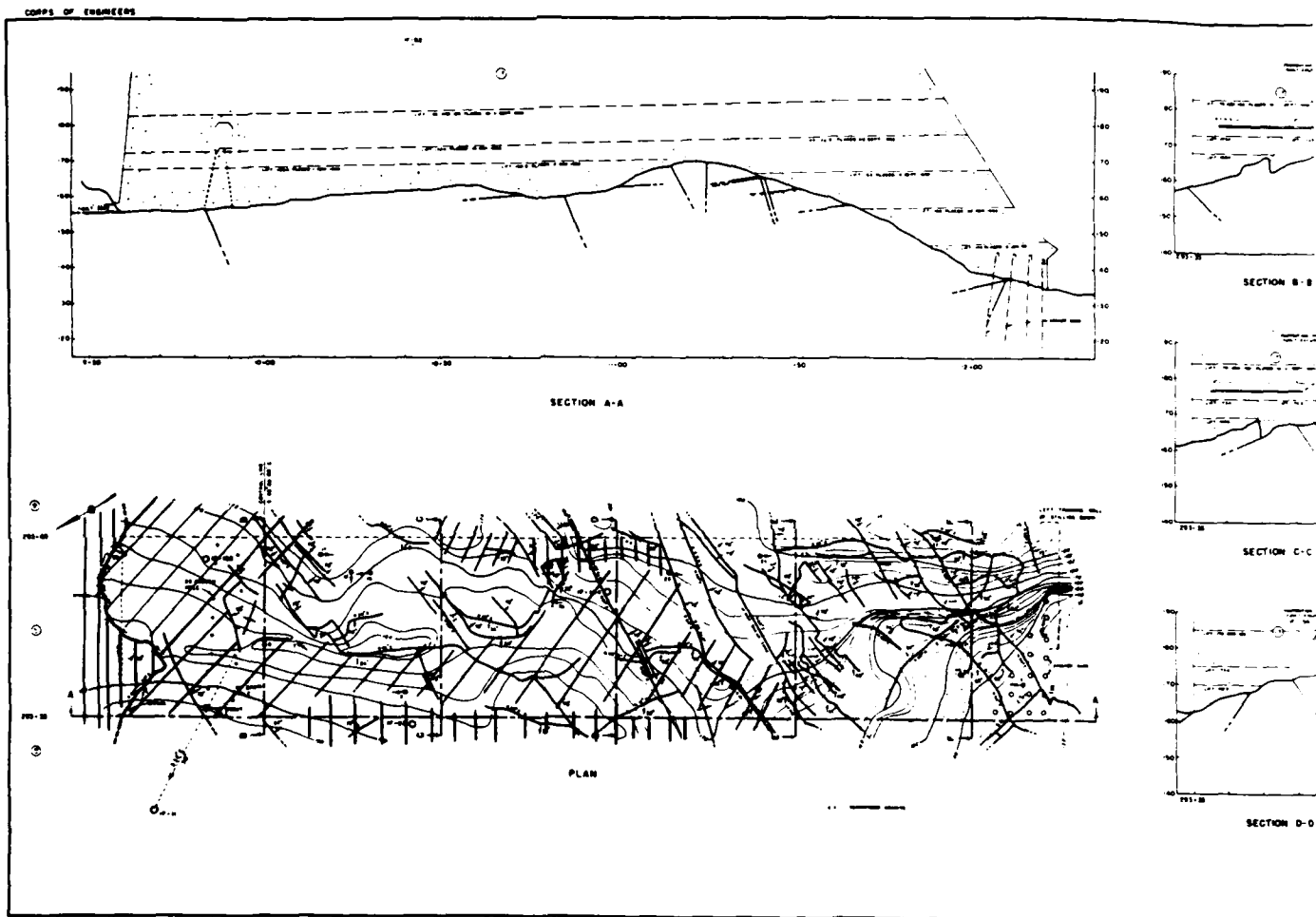and Sections of Concrete Gravity Dam Monolith 14.
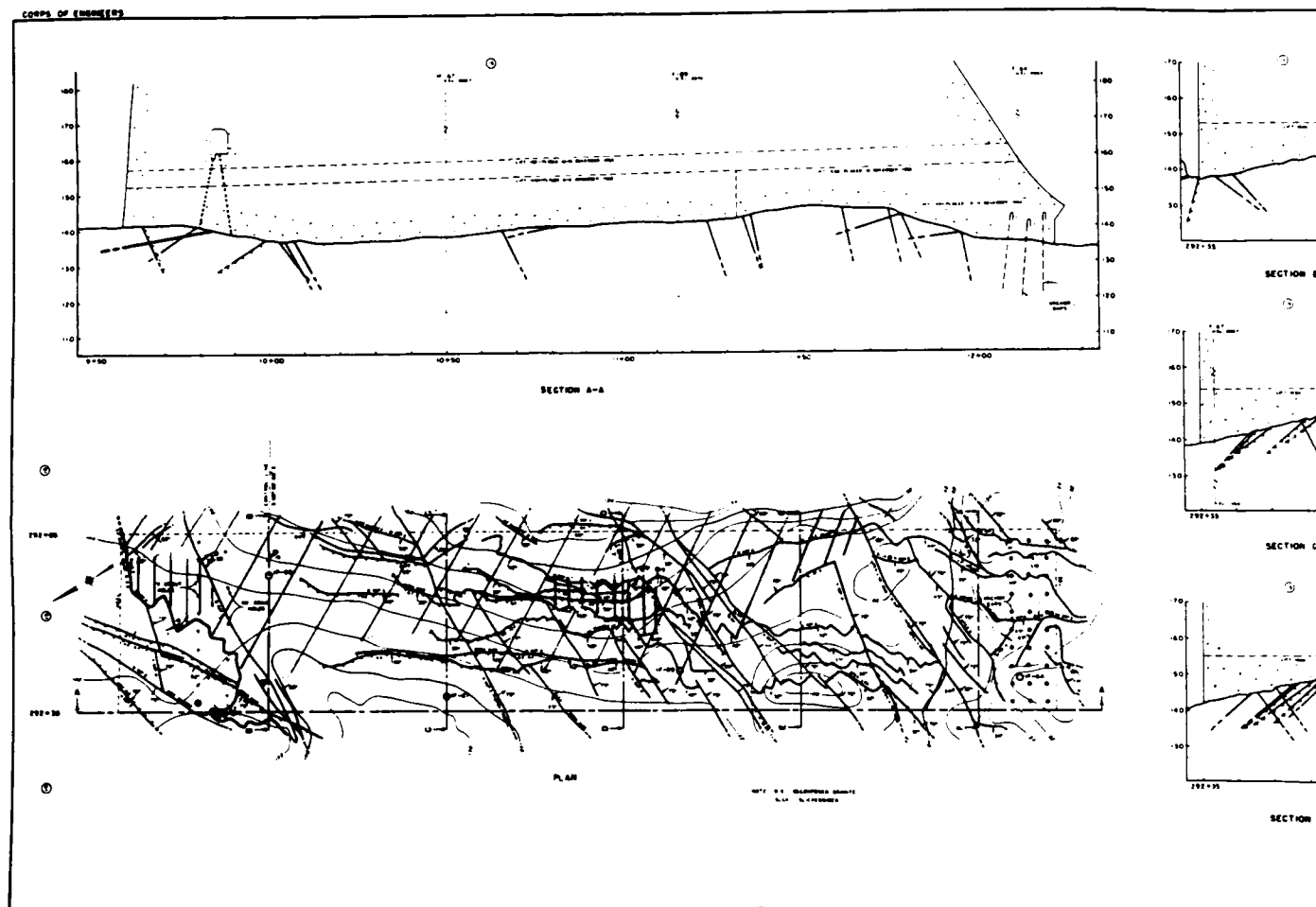
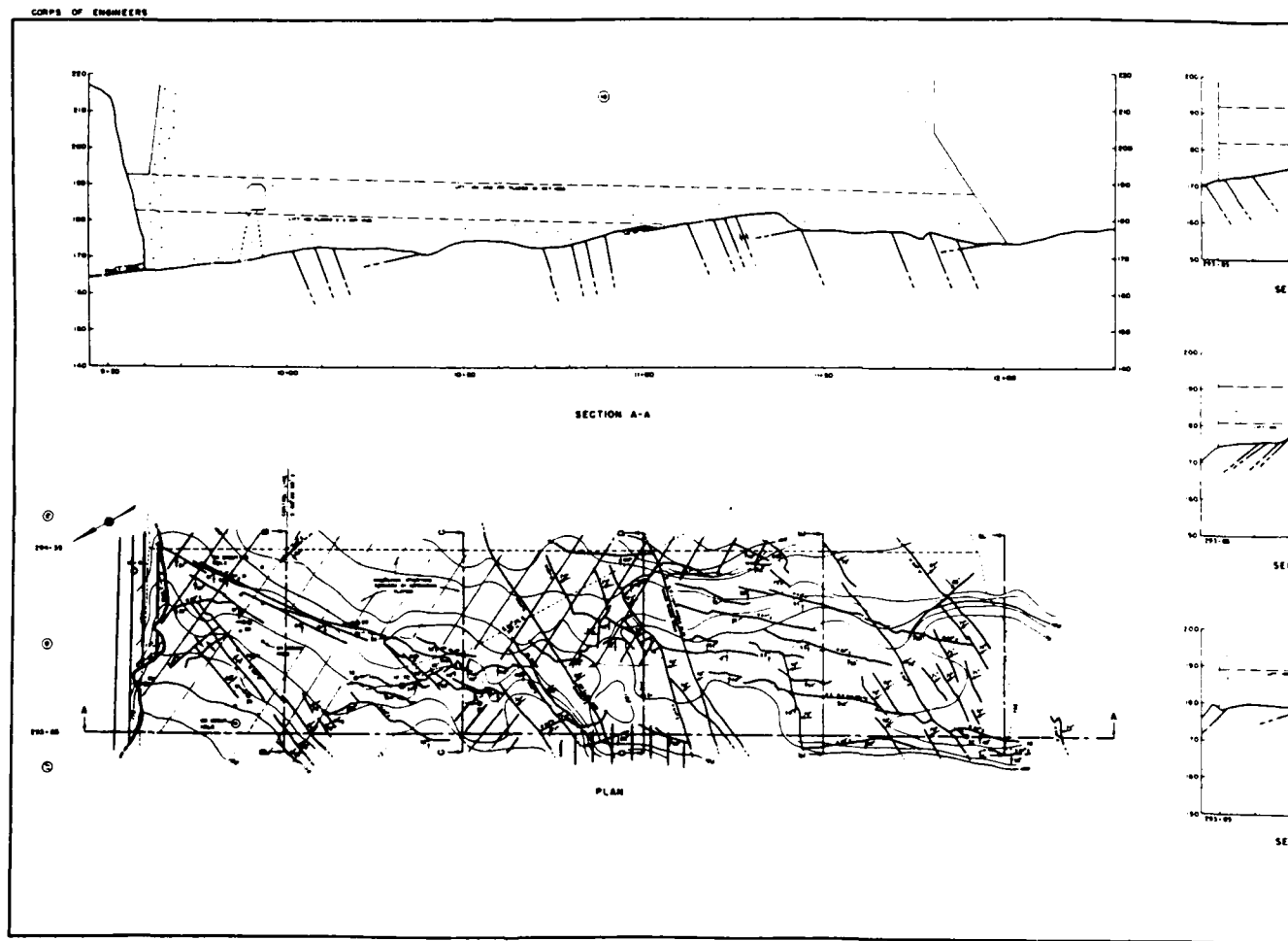**Figure 16. Plan and Sections of Concrete Gravity Dam Monolith 15**

Sections of Concrete Gravity Dam Monolith 15.

Figure 17. Plan and Sections of Concrete Gravity Dam Monoli

id Sections of Concrete Gravity Dam Monolith 16.

SECTION A-A

PLAN

SECTION B-B

SECTION C-C

SECTION D-D

Figure 18. Plan and Sections of Concrete Gravity Dam Monolith

SECTION B-B SECTION E-E

SECTION C-C

SECTION D-D

SECTION F-F

EXPLANATION

FOLSOM PROJECT
AMERICAN RIVER CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 17
PLAN AND SECTIONS

l Sections of Concrete Gravity Dam Monolith 17.

SECTION A-A

PLAN

Figure 19. Plan and Sections of Concrete Gravity Dam M

SECTION B-B

SECTION E-E

SECTION C-C

SECTION F-F

SECTION D-D

EXPLANATION

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 18
PLAN AND SECTIONS

SCALE

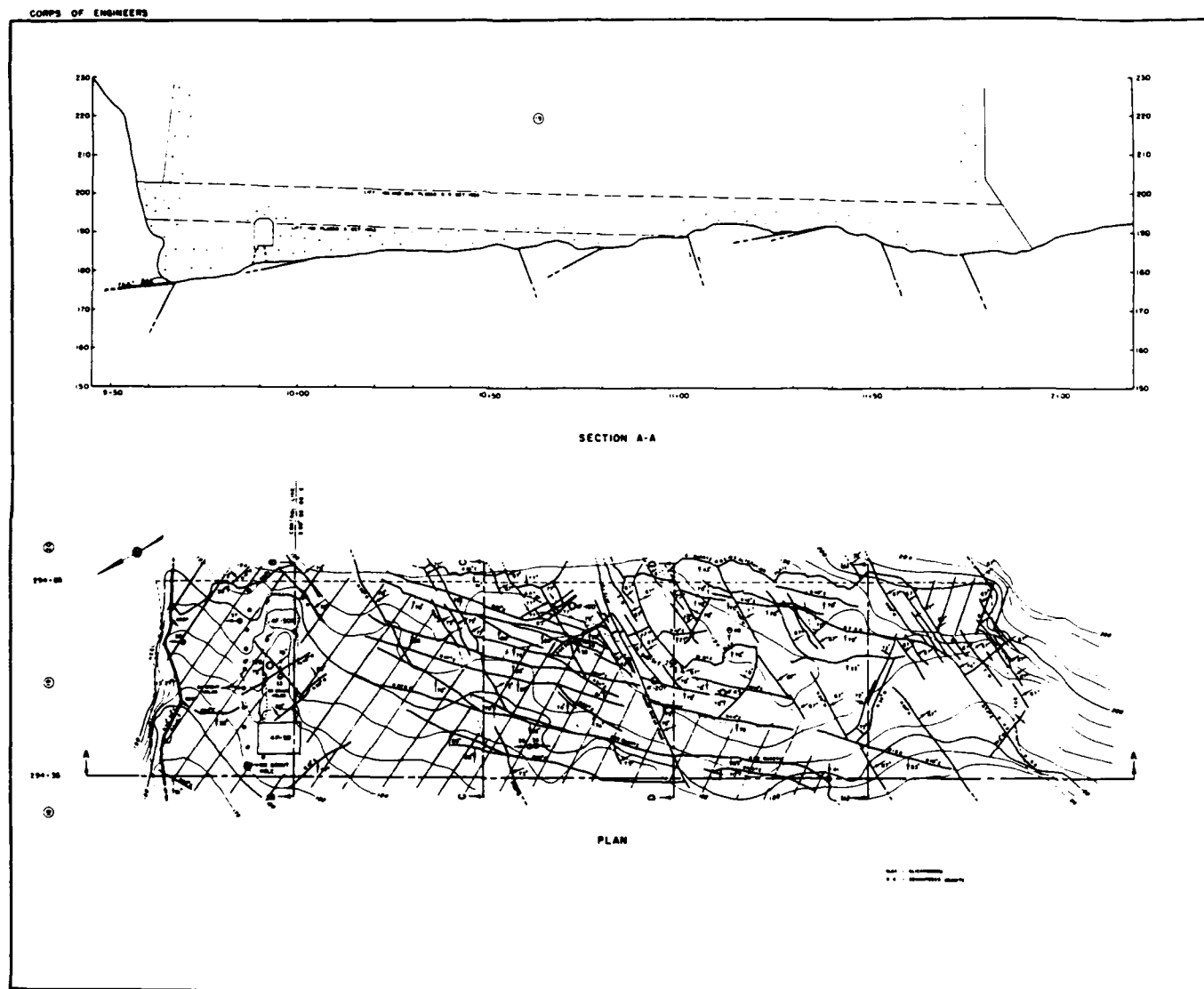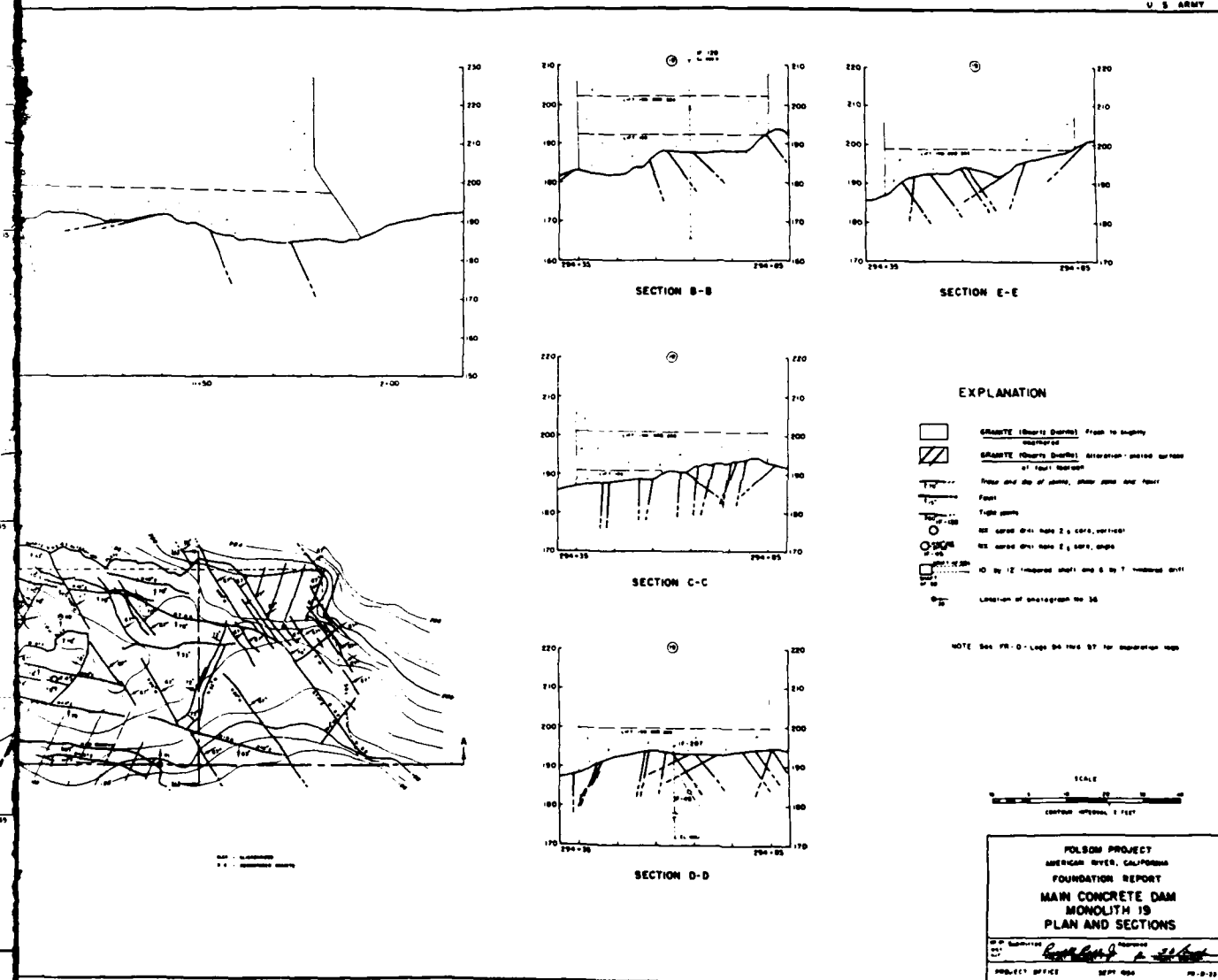IIt**n and Sections of Concrete Gravity Dam Monolith 18.**

1

2

SECTION A-A

PLAN

Figure 20. Plan and Sections of Concrete Gravity Dam Monol

d Sections of Concrete Gravity Dam Monolith 19.

SECTION B-B

SECTION E-E

SECTION C-C

SECTION D-D

EXPLANATION

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
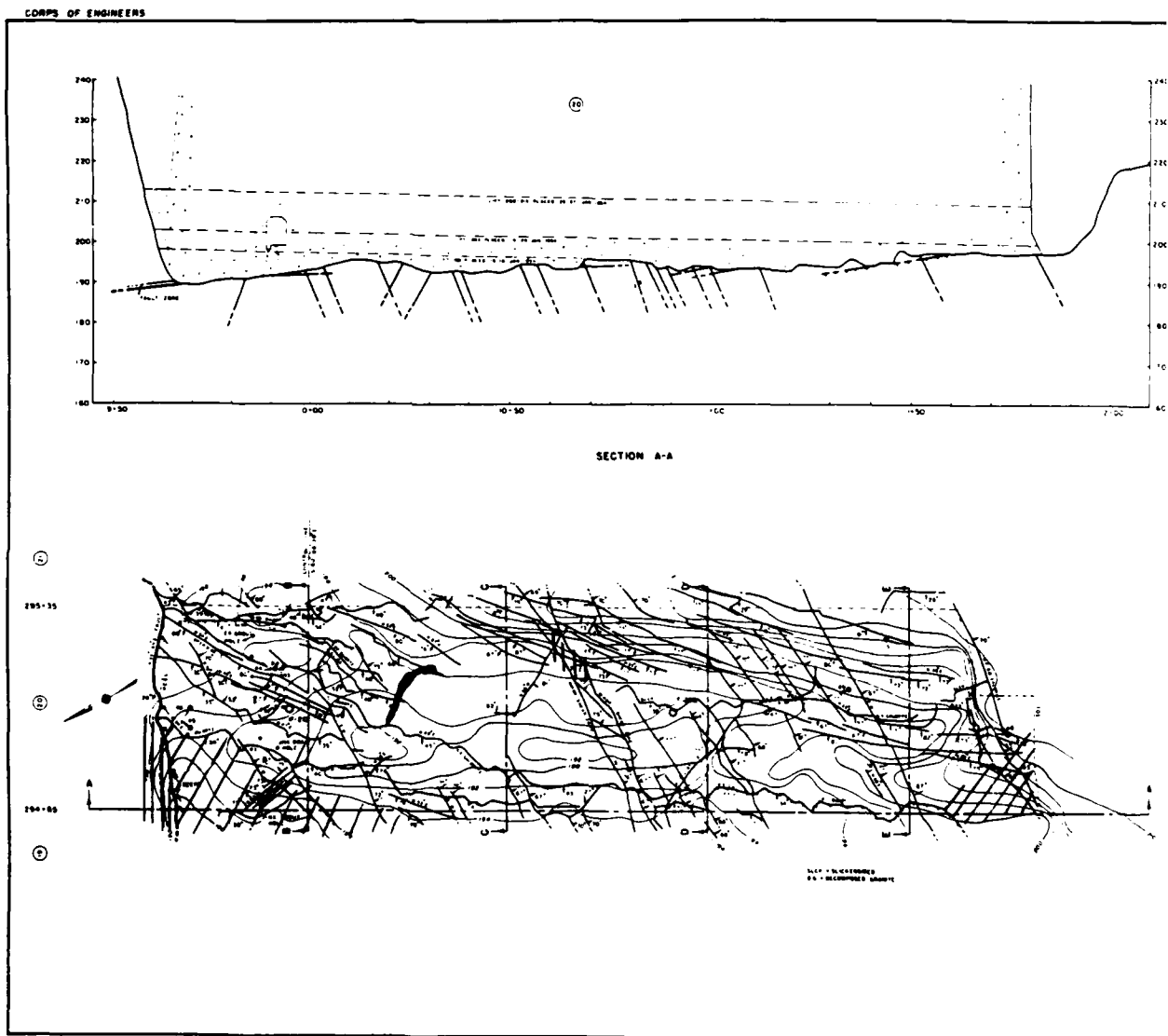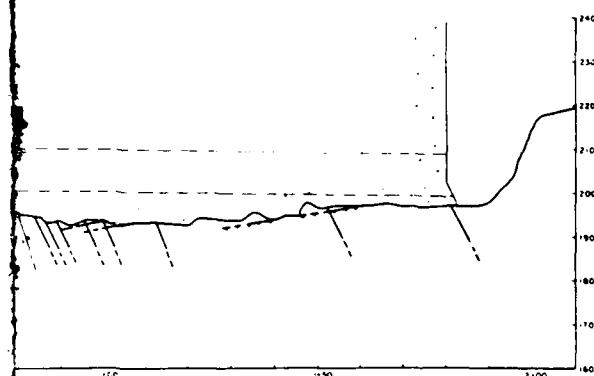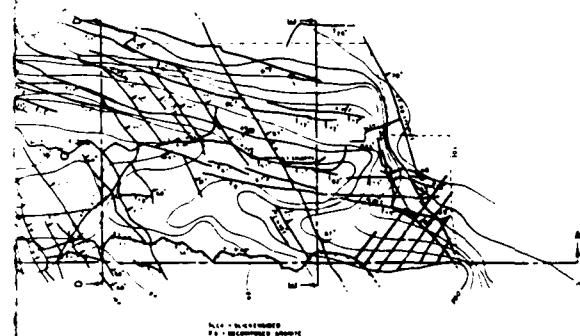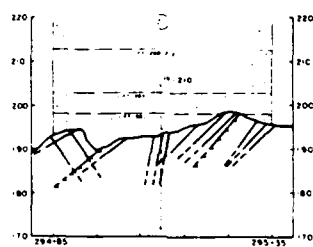MAIN CONCRETE DAM
MONOLITH 19
PLAN AND SECTIONS

SECTION A-A

Figure 21. Plan and Sections of Concrete Gravity Dam

SECTION B-B
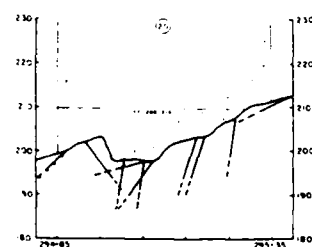
SECTION E-E
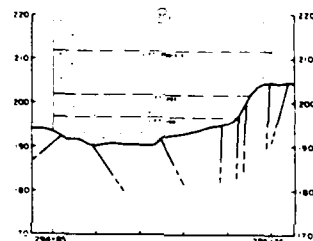
SECTION C-C
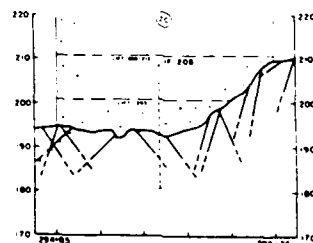
SECTION D-D

SECTION A-A

EXPLANATION

GRANITE (Quartz Diorite) Fresh to slightly weathered

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Alteration-pitted surface of fault septum

MICRODIORITE Dike

Trace and dip of joints, shear zone and fault

Fault

Tight joints

Altered joints striking N 40°-55°E. Small figures indicate alteration effort

NX cored drill hole 2½ core vertical

Location of photograph No 48

NOTE See Fnd Log 98 and 99 exploration logs
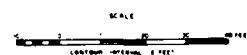
SCALE

CONTOUR INTERVAL 5 FEET

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA

FOUNDATION REPORT

MAIN CONCRETE DAM
MONOLITH 20
PLAN AND SECTIONS

PROJECT OFFICE   SEPT 1954

lan and Sections of Concrete Gravity Dam Monolith 20.
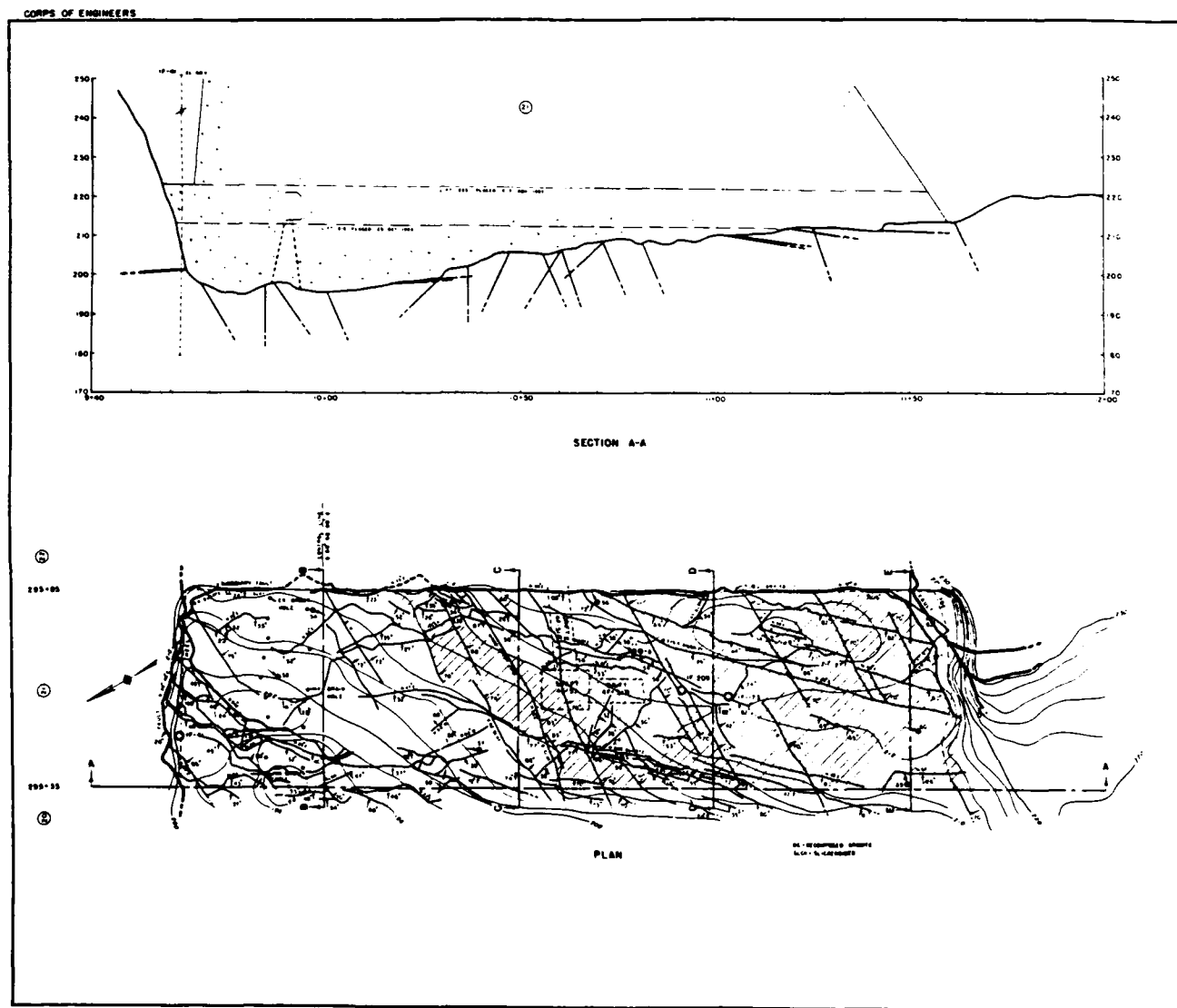
SECTION A-A

PLAN

Figure 22. Plan and Sections of Concrete Gravity Dam M
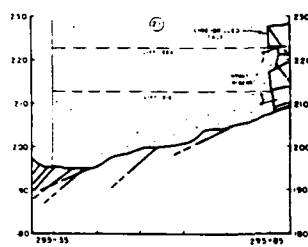
SECTION B-B

SECTION E-E

SECTION C-C

SECTION D-D

EXPLANATION

SCALE

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 21
PLAN AND SECTIONS

nd Sections of Concrete Gravity Dam Monolith 21.

1

2

ELEVATION A-A
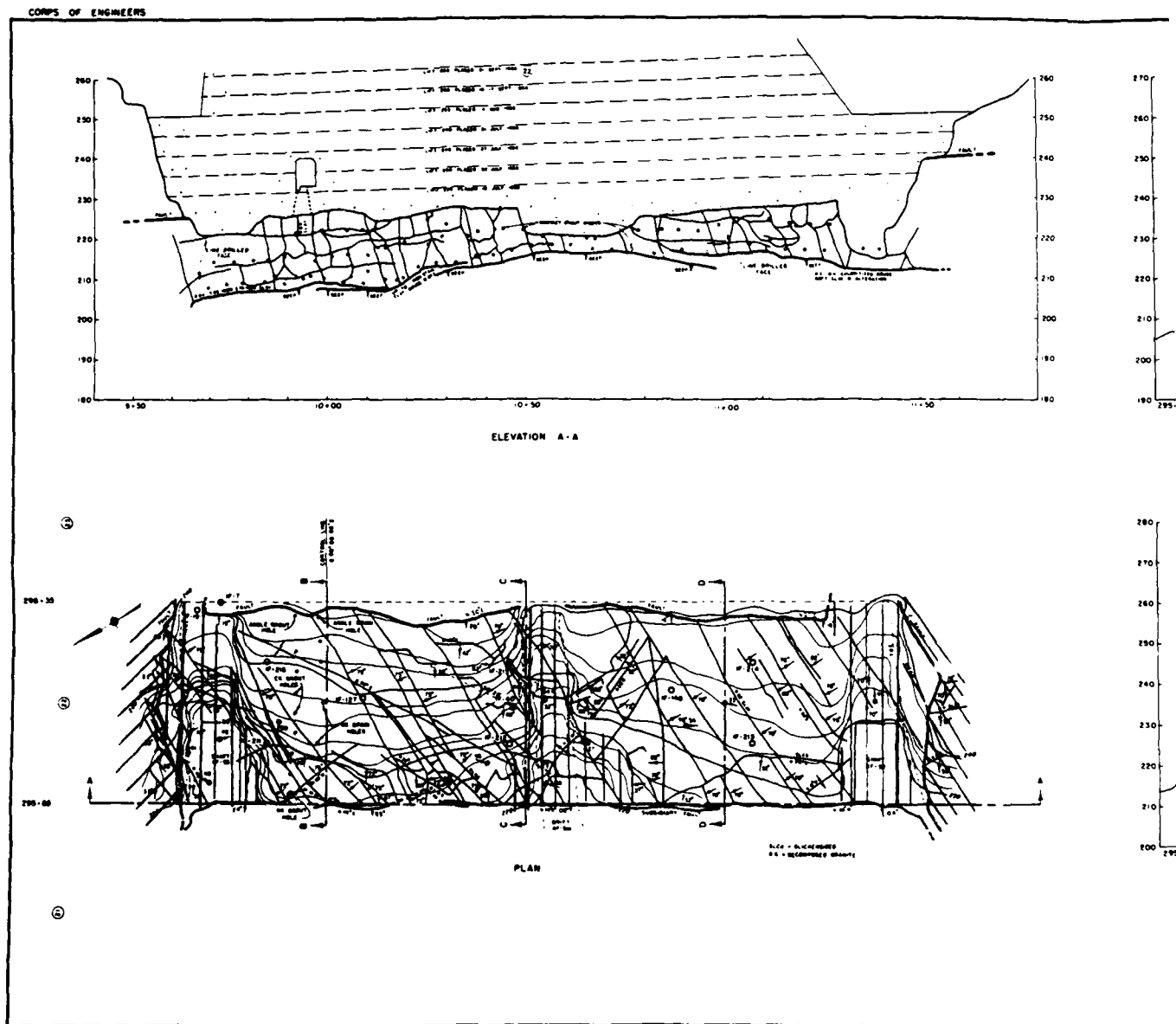
PLAN

Figure 23. Plan and Sections of Concrete Gravity Dam Monolith
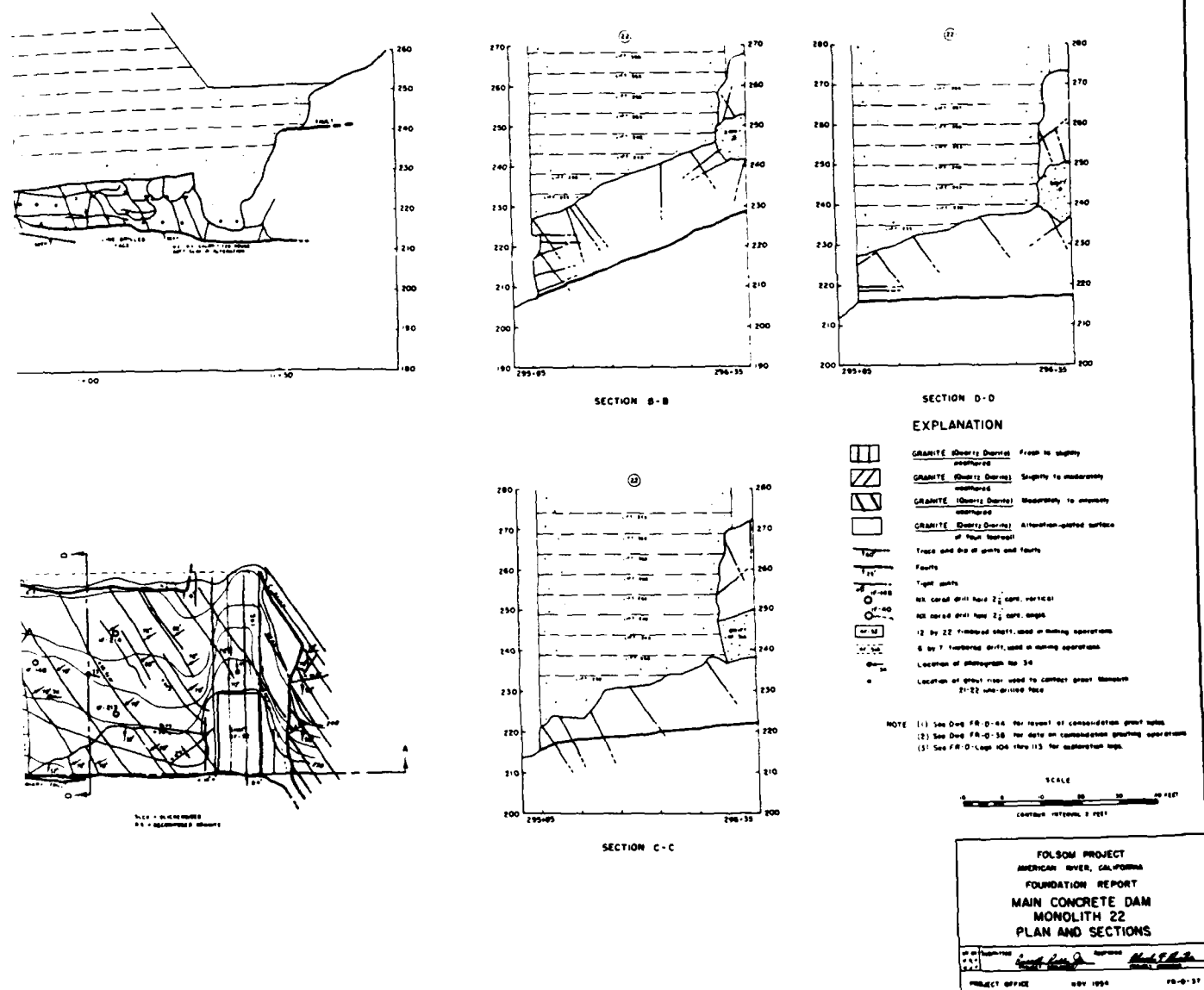
Sectíons of Concrete Gravity Dam Monolith 22.
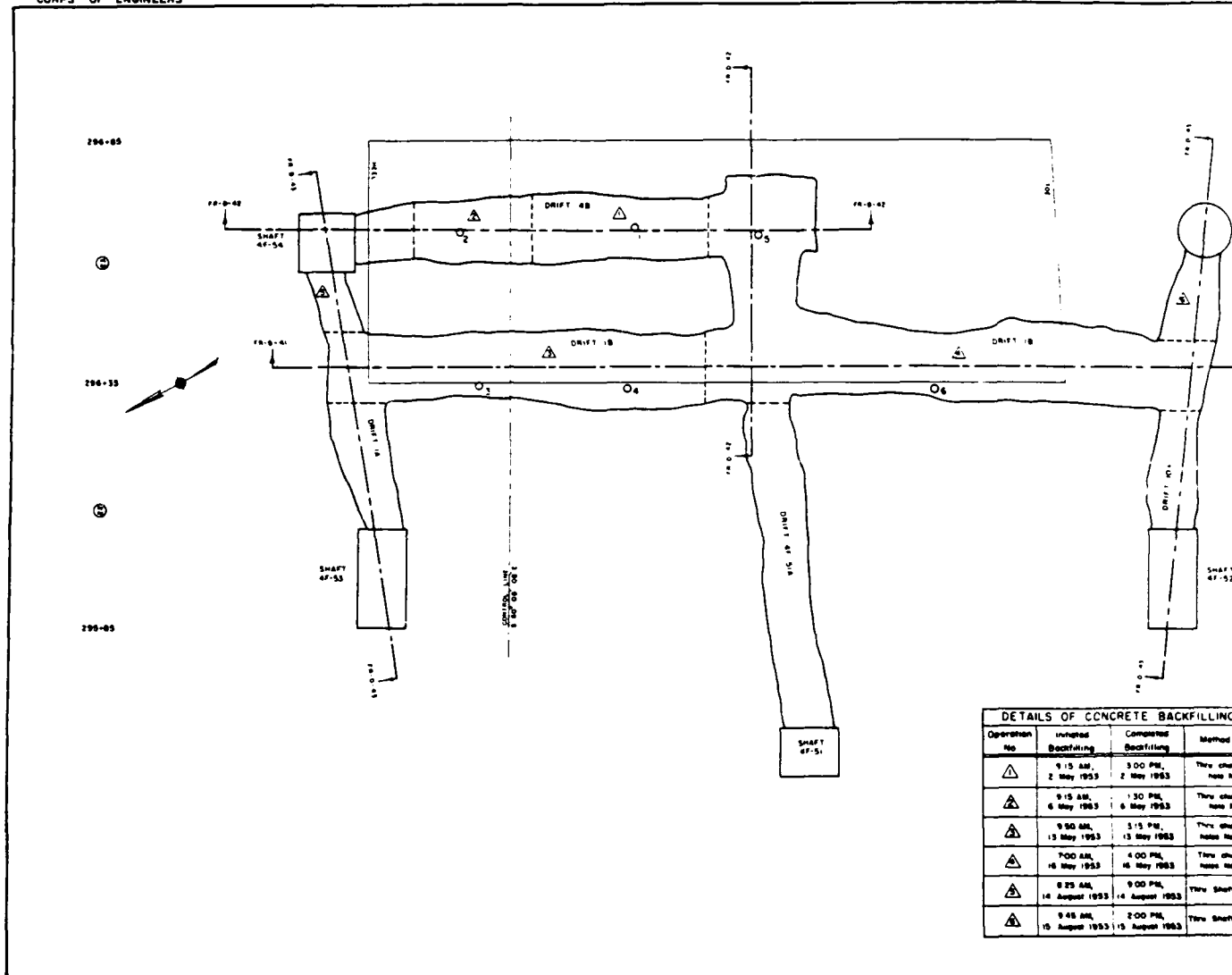
Figure 24. Layout of Mining Operations Concrete Gravity Dam Monolith

EXPLANATION

⚠ Backfill operation, number designates sequence

○³ 10" Churn drill hole

Section of Drift 48, number designates drawing on which section appears

Borrowed for conforming concrete

NOTE Drawings FR-D-41, 42, and 43 show sectional details of drifts and shafts

SHAFT 4F-55

DRIFT 48

DRIFT 46

SHAFT 4F-52

SHAFT 4F-51

DETAILS OF CONCRETE BACKFILLING OPERATIONS			
Operation No.	Initiated Backfilling	Completed Backfilling	Method Used	Cu Yds Concrete
⚠	9:15 A.M., 2 May 1953	3:00 P.M., 2 May 1953	Thru churn drill hole No. 1	184
⚠	9:15 A.M., 6 May 1953	1:30 P.M., 6 May 1953	Thru churn drill hole No. 2	120
⚠	9:00 A.M., 13 May 1953	3:15 P.M., 13 May 1953	Thru churn drill holes Nos. 3, 4	474
⚠	7:00 A.M., 16 May 1953	4:00 P.M., 16 May 1953	Thru churn drill holes Nos. 5, 6	900
⚠	8:25 A.M., 14 August 1953	9:00 P.M., 14 August 1953	Thru Shaft 4F-54	354
⚠	9:45 A.M., 18 August 1953	2:00 P.M., 18 August 1953	Thru Shaft 4F-55	232
			Total	2264

SCALE

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITHS 22-23
LAYOUT OF MINING OPERATIONS

PROJECT OFFICE   NOV 1954   FR-D-40

ning Operations Concrete Gravity Dam Monoliths 22-23.
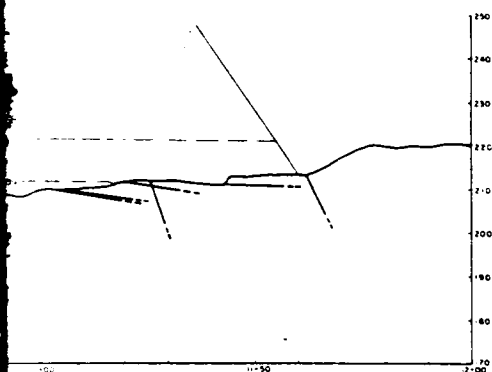
2

ELEVATION A-A

PLAN

Figure 25. Plan and Sections of Concrete Gravity Dam Monoli

SECTION B-B

SECTION D-D

SECTION C-C

EXPLANATION

NOTES

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 23
PLAN AND SECTIONS
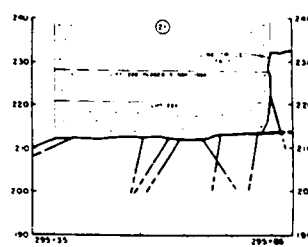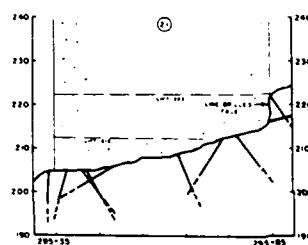
nd Sections of Concrete Gravity Dam Monolith 23.
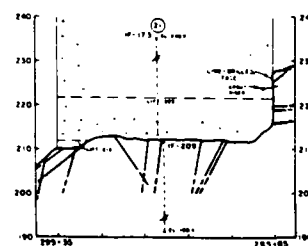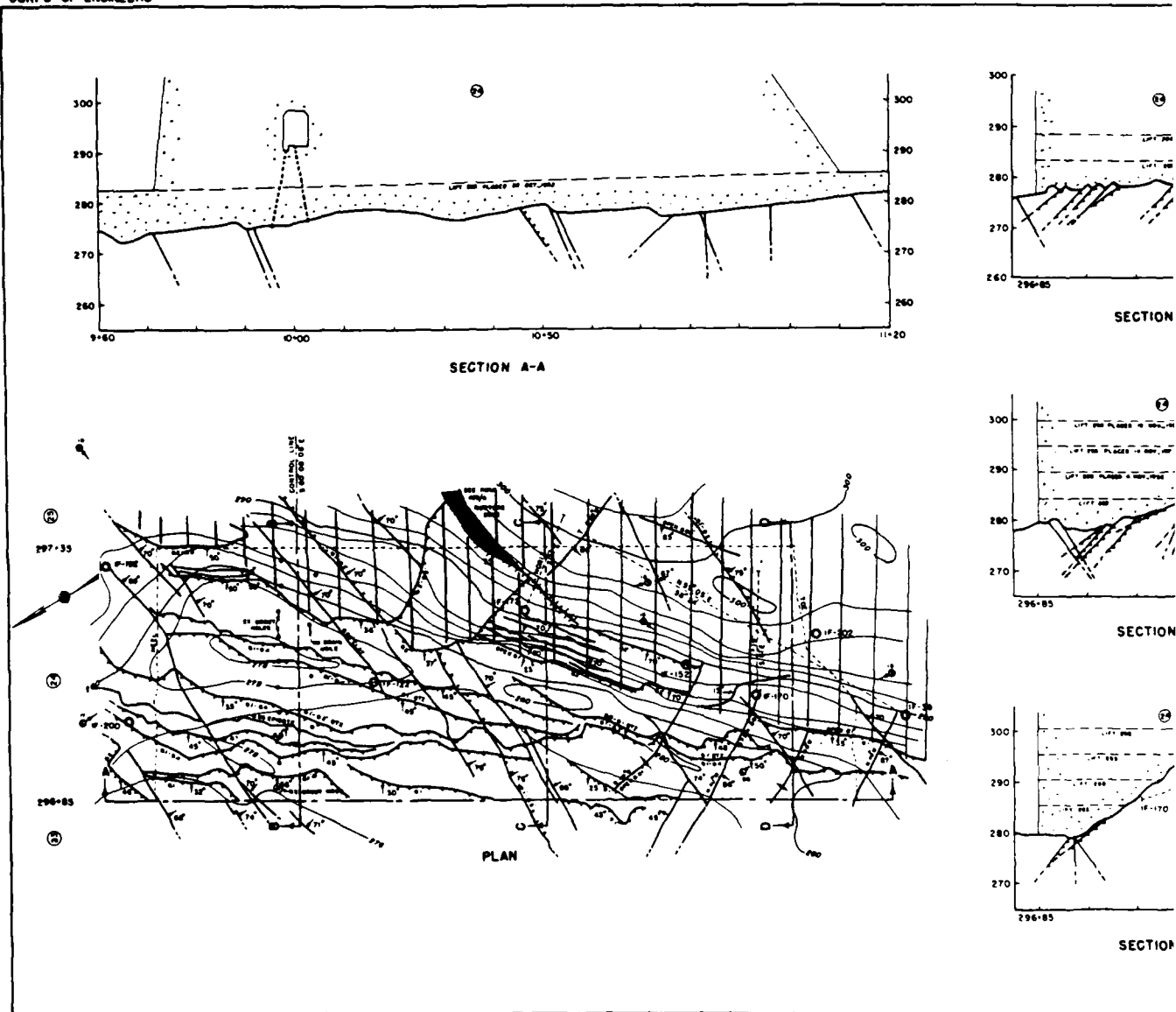
Figure 26. Plan and Sections of Concrete Gravity Dam Mono

SECTION B-B

SECTION C-C

SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite). Fresh to slightly weathered

GRANITE (Quartz Diorite). Slightly to moderately weathered

GRANITE (Quartz Diorite). Moderately to intensely weathered

Trace and dip of joints

Tight joint

Altered joints striking N 40°-55°E. Small figures indicate alteration extent.

Foliation-iomed joints striking N 75°-90°E. Small figures indicate extent of linear estabation.

NX cored drill hole 2⅛" core, vertical

NX cored drill hole 2⅛" core, angle.

Location of photograph No. 20

NOTE   See FR-D-Log 11 thru 18 for exploration logs.

SCALE

CONTOUR INTERVAL 2 FEET

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 24
PLAN AND SECTIONS

Submitted   Approved

PROJECT OFFICE   February 1954   FR-D-7

24

Sections of Concrete Gravity Dam Monolith 24.

SECTION A-A

PLAN

Figure 27. Plan and Sections of Concrete Gravity
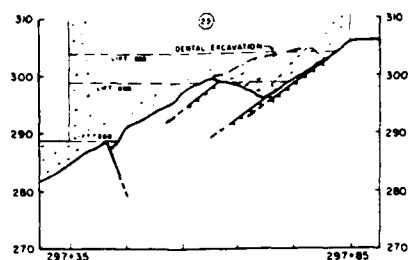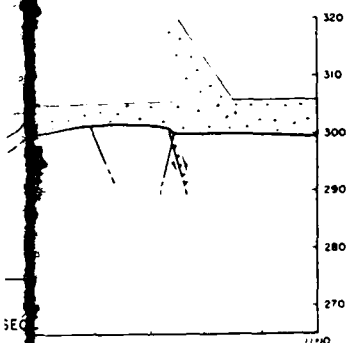
SECTION B-B
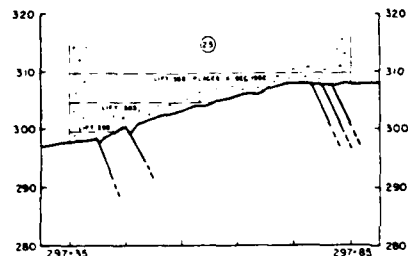
SECTION C-C
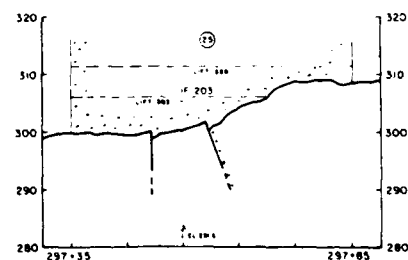
SECTION D-D

EXPLANATION

GRANITE (Quartz Diorite) Fresh to slightly weathered

GRANITE (Quartz Diorite) Slightly to moderately weathered

GRANITE (Quartz Diorite) Moderately to intensely weathered

Trace and dip of joints

Tight joint

Altered joints striking N 40°-55°E Small figures indicate alteration extent

Foliation-lined joints striking N 75-90°E Small figures indicate extent of linear extension

NX cored drill hole 2⅛ core, vertical

NX cored drill hole 2⅛ core, angle

Location of photograph No 11

NOTE   See FR-D - Log 19 thru 23 for exploration logs
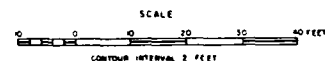
SCALE

CONTOUR INTERVAL 2 FEET

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA

FOUNDATION REPORT

MAIN CONCRETE DAM
MONOLITH 25
PLAN AND SECTIONS

Submitted   Approved

PROJECT OFFICE   February 1954   FR-D-8

Plan and Sections of Concrete Gravity Dam Monolith 25.
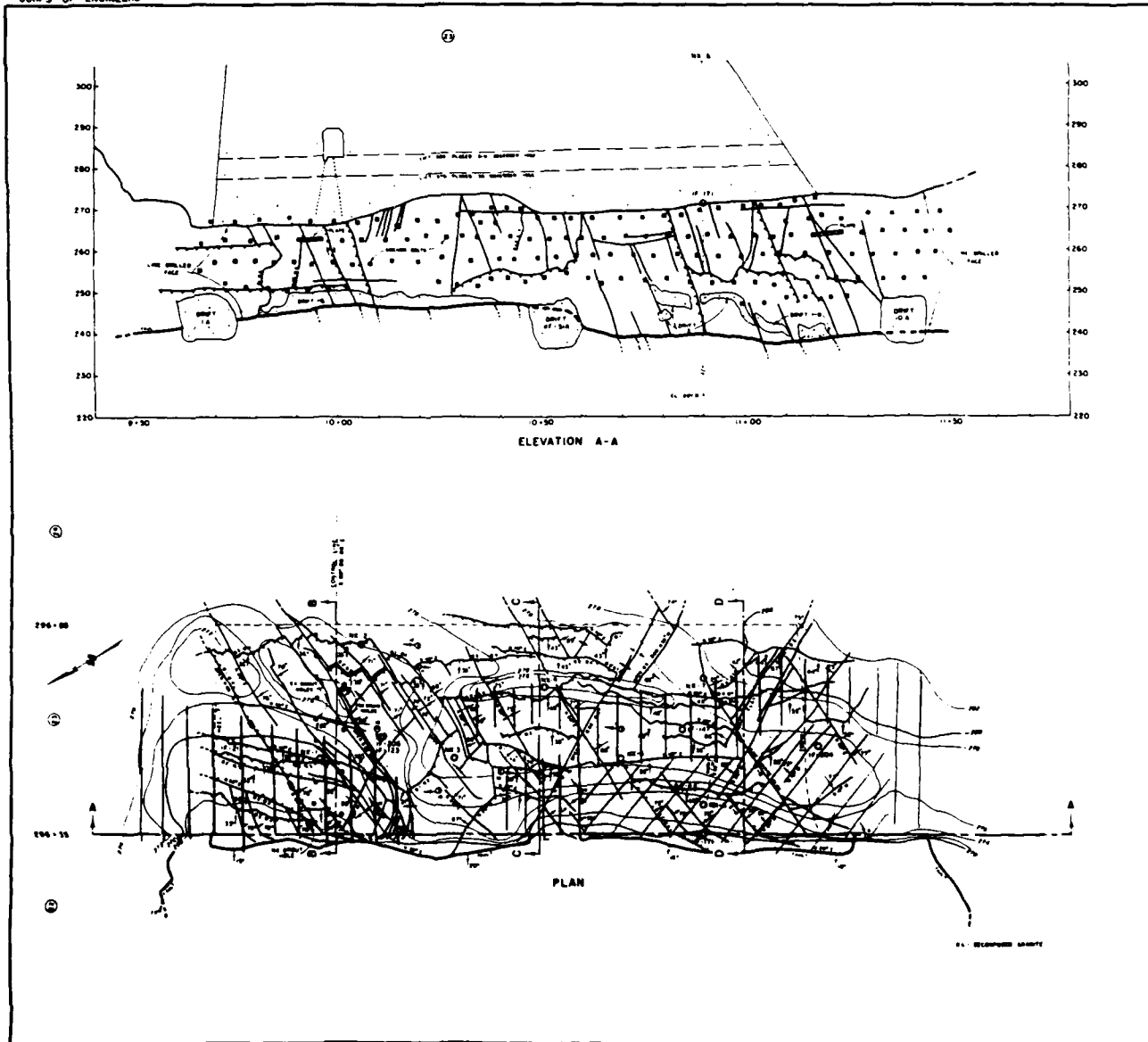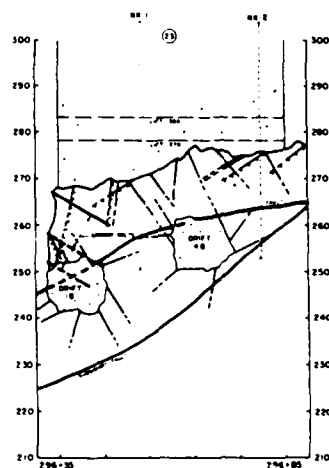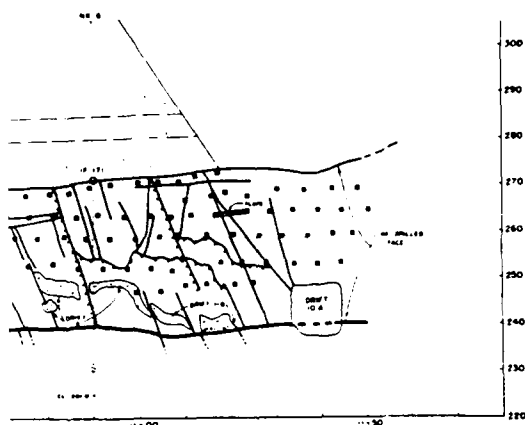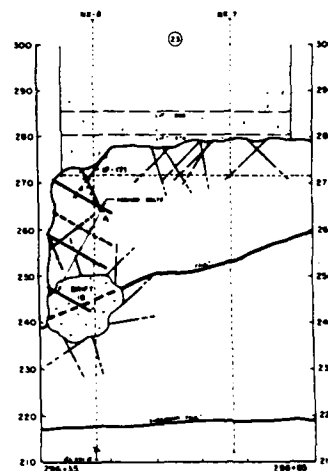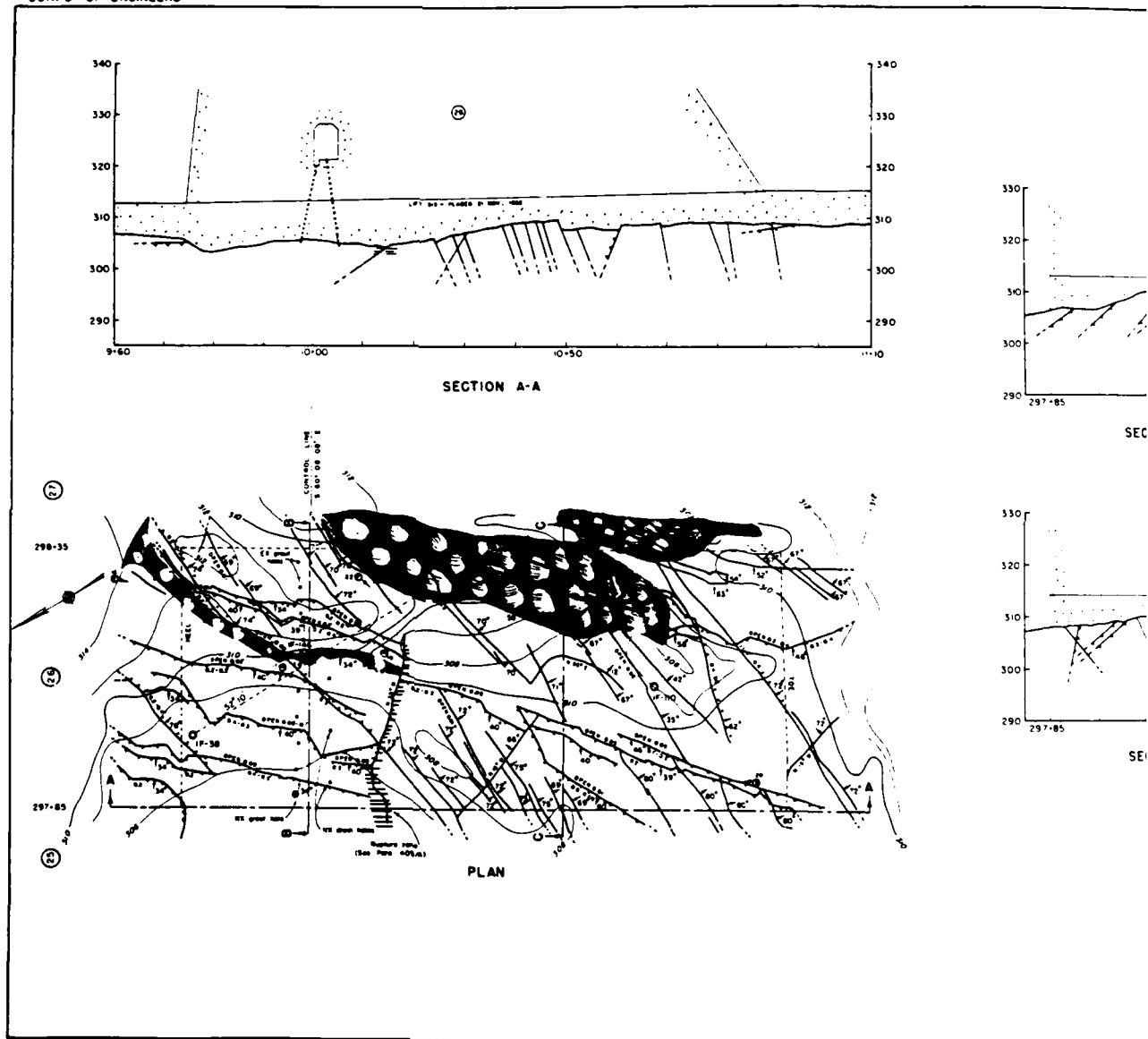
SECTION A-A

PLAN

Figure 28. Plan and Sections of Concrete Gravity Dam Mono
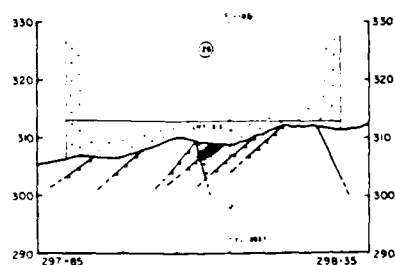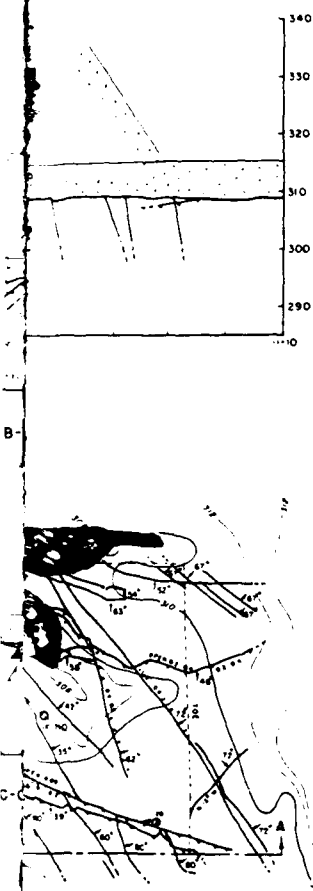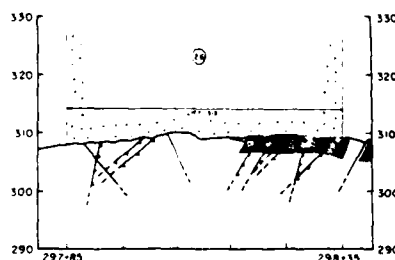
SECTION B-B

SECTION C-C

EXPLANATION

GRANITE, (Quartz Diorite). Slightly to moderately weathered.

GRANITE, (Quartz Diorite). Moderately to intensely weathered.

Trace and dip of joints.

Tight joint.

Altered joints striking N 40°-55° E. Small figures indicate alteration extent.

Foliation-lined joints striking N 75°-90° E. Small figures indicate extent of linear exfoliation.

NX cored drill hole, 2⅛" core, vertical.

NX cored drill hole, 2⅛" core, angle.

Location of photograph No. 22.

NOTE   See FR-D-Log 8 thru 10 for exploration logs.
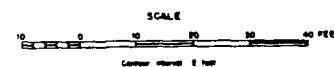
SCALE

10   0   10   20   30   40 FEET

Contour interval 2 feet

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA

FOUNDATION REPORT

MAIN   CONCRETE   DAM
MONOLITH 26 — PLAN & SECTIONS

| Submitted	Approved
PROJECT GEOLOGIST	PROJECT ENGINEER
Project Office	January, 1954	FR-D-3

2nd Sections of Concrete Gravity Dam Monolith 26.
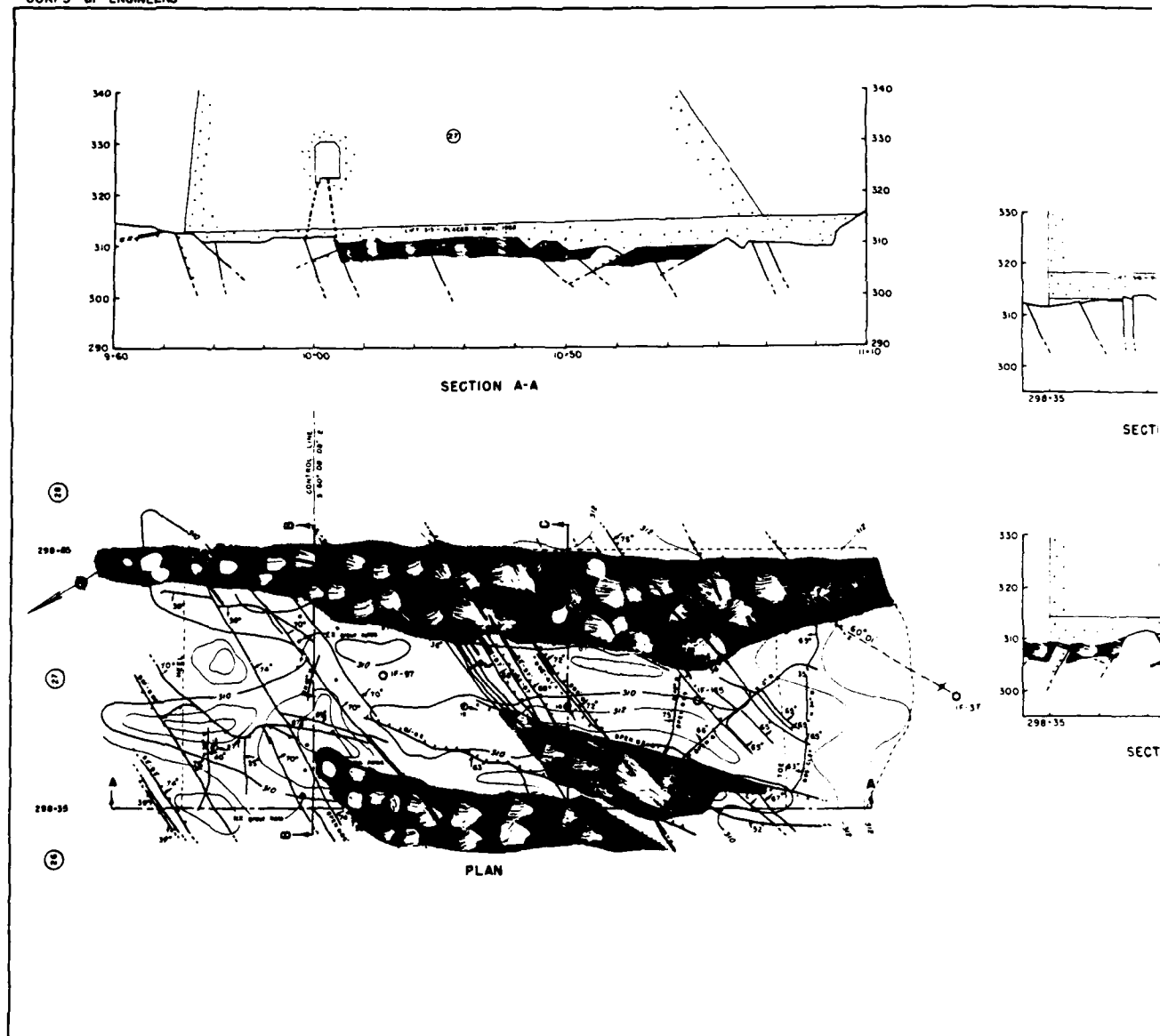
SECTION A-A

PLAN

Figure 29. Plan and Sections of Concrete Gravity Dam Mo
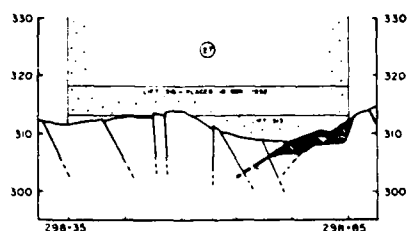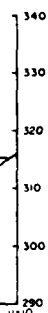
SECTION B-B
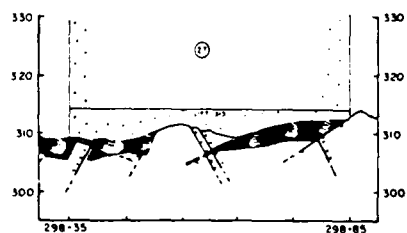
SECTION C-C

EXPLANATION

GRANITE (Quartz Diorite). Slightly to moderately weathered

GRANITE (Quartz Diorite). Moderately to intensely weathered

Trace and dip of joints and fault

Tight joint.

Altered joints strking N 40°-55° E. Small figures indicate alteration extent

Foliation-lined joints strking N 75°-90° E. Small figures indicate extent of linear exfoliation

Fault.

NX cored drill hole, 2⅛ core, vertical.

NX cored drill hole, 2⅛ core, angle.

Location of photograph No. 15.

NOTE   See FR-D- Log 5 thru 7 for exploration logs.
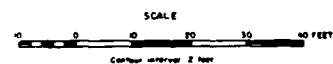
SCALE

Contour interval 2 feet

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA

FOUNDATION REPORT

MAIN CONCRETE DAM
MONOLITH 27 – PLAN & SECTIONS

Submitted   Approved

Project Office   January, 1954   FR-D-4

nd Sections of Concrete Gravity Dam Monolith 27.

2
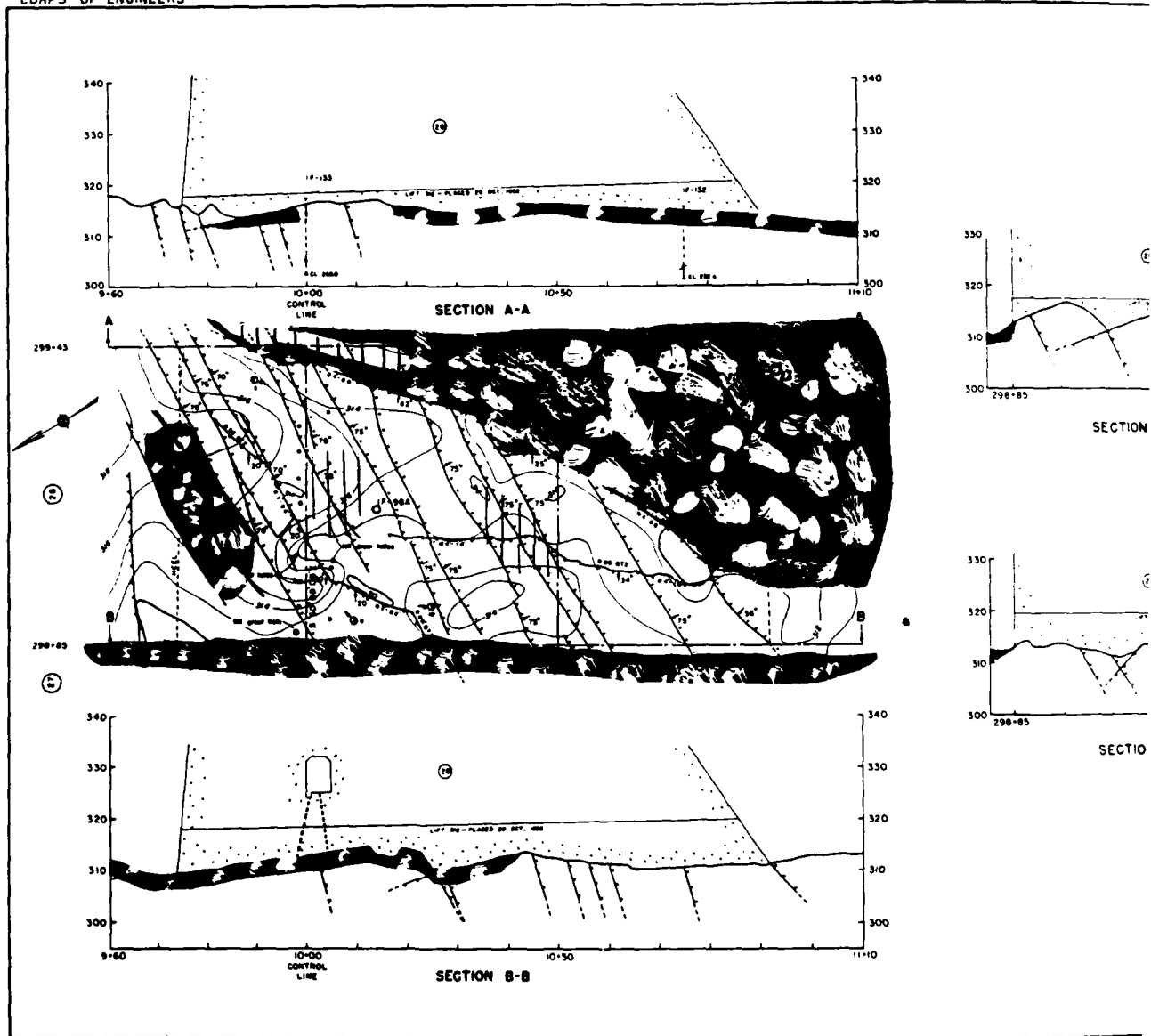
Figure 30. Plan and Sections of Concrete Gravity Dam Mon
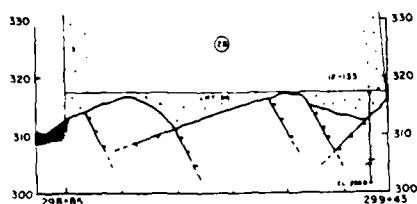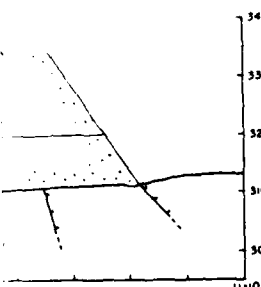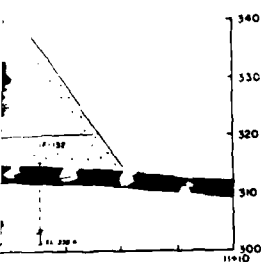
SECTION C-C
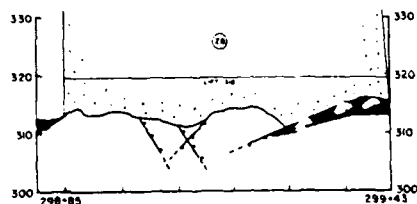
SECTION D-D

SCALE

FOLSOM PROJECT
AMERICAN RIVER, CALIFORNIA
FOUNDATION REPORT
MAIN CONCRETE DAM
MONOLITH 28 – PLAN & SECTIONS

Submitted

Approved

Project Office   January, 1954   FR-D-3

...d Sections of Concrete Gravity Dam Monolith 28.

APPENDIX B

SUMMARY OF WOODWARD-CLYDE REPORT

APPENDIX B: SUMMARY of WOODWARD-CLYDE REPORT

1. The purpose of the initial drilling and testing investigations was
to provide the US Army Engineer District, Sacramento, with representative
foundation rock and concrete core samples for laboratory testing and to obtain
both static and dynamic elastic properties for use in the analyses of the
dam. The Goodman Jack was used to perform in situ testing of the foundation
rock to obtain rock mass deformation properties. The scope of the work was:

    a. To obtain 6-in.-diameter concrete core samples from the dam
       suitable for testing.

    b. To obtain NX-size rock core samples from the foundation of the
       dam suitable for testing.

    c. To determine in-site foundation rock mass deformation
       properties.

    d. To prepare a written report summarzing, in detail, the field
       investigations and descriptions of concrete and the foundation.

2. According to design specifications, the dam has a 2- to 10-ft-thick
shell of rich concrete (high cement content) which has a maximum aggregate
size of 6 in. The interior concrete is lean concrete (less cement content)
having a maximum aggregate size also of 6 in. Therefore, both concretes were
expected to be similar in appearance. In order to guarantee testing of lean
concrete, the concrete hole on the downstream face of the dam was drilled a
minimum of 10 ft through the exterior, rich concrete, and then drilled a
minimum of 5 ft into the lean concrete. The remaining holes penetrated a
minimum of 5 ft into the interior mass concrete. Precautions were taken to
obtain a minimum of two 30-in.-long or four 15-in.-long samples of the
interior mass concrete from each hole. However, 30-in. samples were not
readily obtained because of breakage during drilling. The breaking occurred
at the aggregate cement-mortar contact. Lengths of core holes and concrete
cores were listed in their report.

3. The condition of the concrete within the dam was determined overall
to be very good. Most of the voids in the concrete cores were observed on the
surface of the samples where pieces of aggregate were dislodged. The only
exception to this was reported in core 1C-20A where voids up to 1/2 in. in
diameter were located. Also, at 7 ft from the collar of this hole, a 3-in.
aggregate was found completely absent of mortar creating a void 2 in. into the

B2

core. However, the cement-mortar aggregate bond within all samples was observed to be tight. The aggregate quality was reported to be generally good, consisting of very hard metamorphic, volcanic, and granitic rocks. The mortar quality was observed to be generally very good, with only a few occurrences of excessive sand found in the binder material. Several core samples did contain 1/2-in. size pieces of wood fragments. The aggregate size was found to be consistent with specifications. The maximum aggregate size varied from a low of 3 in. to a high of 6 in. Where core holes were drilled on the downstream face, the cores were taken through the shell concrete and into the interior mass concrete, and no readily discernable change in aggregate size was noted in the two types of concrete. Aggregate sizes larger than 6 in. were found in several core samples. Almost without exception, when the core sample broke before removal, the core broke along a large piece of aggregate and around several pieces of aggregate at a lift joint.

4. All the core samples from the crest core holes broke at a construction lift joint 1.8 to 2.7 ft from the top of the crest indicating a weak bond across this lift joint. This lift joint surface contained a hard, white, powder-type material that was thought to be calcium carbonate. Brown stains were observed on this joint surface, which were caused by water passing along the joint surface. The only other construction lift joint that was intersected was located on the left abutment near the downstream toe of the dam. The surface of the lift joint was smooth and coated with a hard, white, powder-type material, again thought to be calcium carbonate.

5. The rock recovered in the six NX-size core holes was described as granodiorite or granite for simplicity. The granite was described as a light gray and white speckled, medium to coarse granular, hard to very hard rock. Inspection of the rock cores showed that the granite rock is only slightly weathered near the surface and is generally unweathered below the top few feet of rock. In order to provide an indication of the degree of weathering and fracturing of the rock, the amount of core recovered in each coring was measured. Very weathered rock or highly fractured rock was ground up during drilling and was not recovered. Rock core recovering was very good, varying from 91 percent in TH-C to 100 percent in TH-D and TH-F.

6. The Rock Quality Designation (RQD), an index of core recovery, is used to reflect rock quality with respect to degree of fracturing. RQD values and corresponding degrees of rock quality are given in Table B1. The RQD was

B3

calculated by measuring the total length of pieces of core, 4 in. or longer as a percentage of the length of the total core. As shown in Table B2, the RQD values ranged from 36 (poor rock quality) to 96 (excellent rock quality). The RQD values indicated that the intensity of fracturing in the foundation decreases from below the Right Wing Dam to below the Left Wing Dam. In general, the Woodward-Clyde report confirms the Geotechnical Laboratory (GL) study reported in Appendix A. The GL describes the foundation conditions under each monolith in considerable detail.

7. Goodman Jack tests were performed in the six NX holes to determine insitu modulus values at depths of 10 and 20 ft from the top of the rock. The Goodman Jack test provides the change in pressure from each successive reading and the corresponding change in diameter of an NX (3-in.) borehole. These measurements were used to calculate the static deformation modulus of the rock mass. The deformation modulus is a measure of the amount of strain, which includes both elastic and plastic deformation, the foundation rock will undergo for a given load. The deformation modulus values and corresponding modulus of elasticity values are shown in Table B3. The deformation modulus value was calculated assuming a Poisson's ratio of 0.25, which was selected based on laboratory tests of similar granite. No correlation was found between the deformation modulus of the rock and the orientation of the borehole jack. The deformation modulus did not appear to be a function of RQD but was directly changed with the degree of fracturing. The modulus of deformation was also constant with depth. The modulus of deformation did increase, going from the Right Wing Dam to the Left Wing Dam.

Table B1

RQD Index

RQD Percent	Rock Quality
Less than 25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

TABLE B2

CORE RECOVERY AND RQD - CORE HOLES TH-A TO TH-F

TH-A

Interval ft	Core Recovery %	RQD %
35.3-38.6	100	60
38.6-43.1	100	44
43.1-45.7	100	25
45.7-51.0	87	18
51.0-56.0	92	43
Average	95	36

TH-B

Interval ft	Core Recovery %	RQD %
10.3-11.6	100	50
11.6-15.8	100	86
15.8-18.6	100	93
18.6-21.8	100	100
21.8-25.6	87	68
25.6-30.1	73	22
30.1-32.7	92	38
Average	92	65

TH-C

Interval ft	Core Recovery %	RQD %
12.2-16.1	97	54
16.1-18.1	100	85
18.1-20.1	100	90
20.1-22.4	100	75
22.4-27.5	80	59
27.5-30.2	100	70
30.2-32.2	75	75
Average	91	67

TH-D

Interval ft	Core Recovery %	RQD %
9.6-11.2	100	44
11.2-16.0	100	98
16.0-20.0	100	100
20.0-22.0	100	100
22.0-25.9	100	100
25.9-29.7	100	100
Average	100	96

TH-E

Interval ft	Core Recovery %	RQD %
27.1-30.2	100	94
30.2-31.7	100	100
31.7-36.6	100	100
36.6-40.7	100	100
40.7-40.9	100	0
40.9-42.9	100	100
42.9-46.3	97	85
Average	99	91

TH-F

Interval ft	Core Recovery %	RQD %
13.4-15.6	100	59
15.6-17.9	100	100
17.9-22.6	100	100
22.6-27.5	100	96
27.5-28.4	100	100
28.4-33.2	100	88
Average	100	91

Table B3
Summary of Calculated Modulus of Deformation Values

Borehole/ Test No.	Depth ft	Orientation of NX-jack [1]	Modulus of Deformation psi x 10^6 [2]	Modulus of Elasticity psi x 10^6 [3]
THA-1	54.5	Par.	0.7	1.5
THA-2	52.8	Per.	0.5	1.6
THA-3	45.8	Par.	0.2	0.9
THA-4	44.2	Per.	0.6	0.8
THA-5	38.9	Par.	0.4	1.5
THA-6	37.2	Per.	0.7	0.7
THB-1	32.4	Par.	--- [4]	---
THB-2	30.9	Per.	0.6	1.0
THB-3	21.6	Per.	2.4	3.4
THB-4	20.0	Par.	1.9	2.4
THB-5	12.4	Par.	1.6	---
THB-6	14.4	Per.	0.5	0.6
THC-1	30.0	Per.	1.7	7.5
THC-2	28.4	Par.	--- [4]	---
THC-3	23.0	Par.	2.1	---
THC-4	21.5	Per	1.2	---
THC-5	17.9	Par	1.7	---
THC-6	19.2	Per.	2.5	---
THD-1	29.6	Per.	1.0	1.6
THD-2	27.9	Par.	1.1	1.9
THD-3	19.9	Par.	1.4	---
THD-4	18.2	Per.	1.3	1.5
THD-5	12.3	Per.	1.2	1.6
THD-6	10.6	Par.	0.8	1.4
THE-1	46.0	Per.	1.5	---
THE-2	44.3	Par.	2.4	2.9
THE-3	35.5	Per.	3.1	---
THE-4	33.5	Par	3.9	---
THE-5	30.9	Per.	3.0	---
THE-6	29.3	Par.	1.8	3.2
THF-1	32.8	Per.	3.1	---
THF-2	30.6	Par.	1.8	2.2
THF-3	23.6	Par.	2.6	3.1
THF-4	22.1	Per.	3.0	---
THF-5	16.2	Par.	2.0	2.6
THF-6	14.6	Per.	0.4	2.6

1) Orientation of borehole jack is with respect to the predominant joint set.
    Par. - Parallel to predominant joint set.
    Per. - Perpendicular to predominant joint set.

2) Modulus of deformation is calculated from the linear portion of the first loading cycle.

3) Modulus of Elasticity calculated from the second loading cycle.

4) Test data beyond linear range of Goodman Jack equipment.

APPENDIX C
FOLSOM DAM
CONCRETE CORE TESTING (DSAP)

APPENDIX C

FOLSOM DAM
CONCRETE CORE TESTING
DAM SAFETY ANALYSIS PROGRAM (DSAP)

NOVEMBER 1983

Authorization

1. Work reported herein was requested by DA Form 2544, No. SPKED-F-83-92, dated 14 July 1983.

Purpose

2. The purpose of this study was to provide concrete properties data for the DSAP program on Folsom Dam.

Samples

3. Eighty-seven boxes of concrete cores were received 18 July 1983. The core was cut into as many 12-in. length samples as possible. Forty-five "lean mix" samples and 32 "rich mix" samples were forwarded to the Bureau of Reclamation for testing purposes. SPD Laboratory retained 36 samples for testing purposes. Of these 36 samples, 21 were tested and 15 were not suitable for testing. Mr. John Hess of the Sacramento District selected which samples were to be tested and what tests should be performed in the SPD Laboratory. All of the samples tested in this laboratory were from "lean mix" cores.

Tests

4. Eleven concrete samples were tested in accordance with ASTM C 469, Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression. Ten core samples were tested in accordance with ASTM C 496; Splitting Tensile Strength of Cylindrical Concrete Specimens.

C2

Test Results

5. The test results are reported as follows:

    <u>a</u>. Table C1, Summary of Testing on Concrete Cores from Folsom Dam.

    <u>b</u>. Table C2, Splitting Tensile Strengths of Cylindrical Concrete Specimens.

    <u>c</u>. Laboratory testing machine printouts; ten printouts.

Table C1

Summary of Testing* on Concrete Cores from Folsom Dam

Laboratory Sample No.	Unconfined Compressive Strength, σ, (psi)	Modulus of Elasticity, E $(\times 10^6 \text{ psi})$	Poisson's Ratio
FL-1	3,780	4.730	0.188
FL-2	5,480	5.220	--
FL-5	3,710	4.126	--
FL-6	4,710	4.638	--
FL-8	4,600	5.108	--
FL-9	2,830	3.328	0.294
FL-10	4,630	3.860	--
FL-11	4,920	3.782	0.173
FL-12	2,940	3.354	0.171
FL-15	4,990	4.336	0.130
FL-16	4,170	3.478	0.188

* Splitting tensile strengths are shown in Table C2.

Table C2

Splitting Tensile Strengths of Cylindrical Concrete Specimens

(ASTM C 496)

Laboratory Sample No.	Tensile Strength (psi)
FL-3	255
FL-4	415
FL-7	620
FL-13	--
FL-14	470
	460
	500
	305
	310
	575
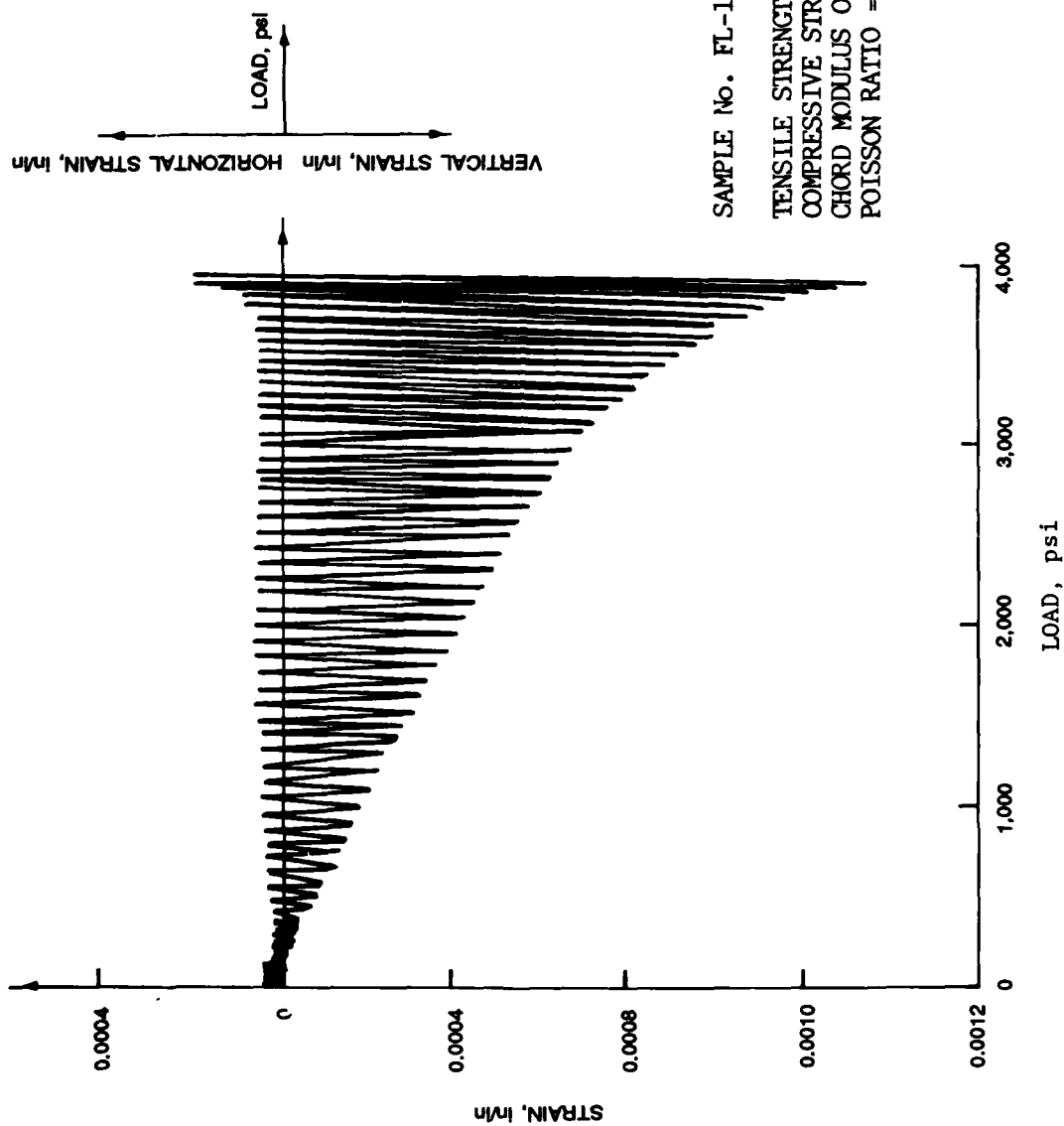
SAMPLE No. FL-1

TENSILE STRENGTH = NOT AVAILABLE
COMPRESSIVE STRENGTH = 3784 psi
CHORD MODULUS OF ELASTICITY = 2.365E + 06
POISSON RATIO = 0.188

Figure C.1 Concrete Test Data for Sample No. FL-1
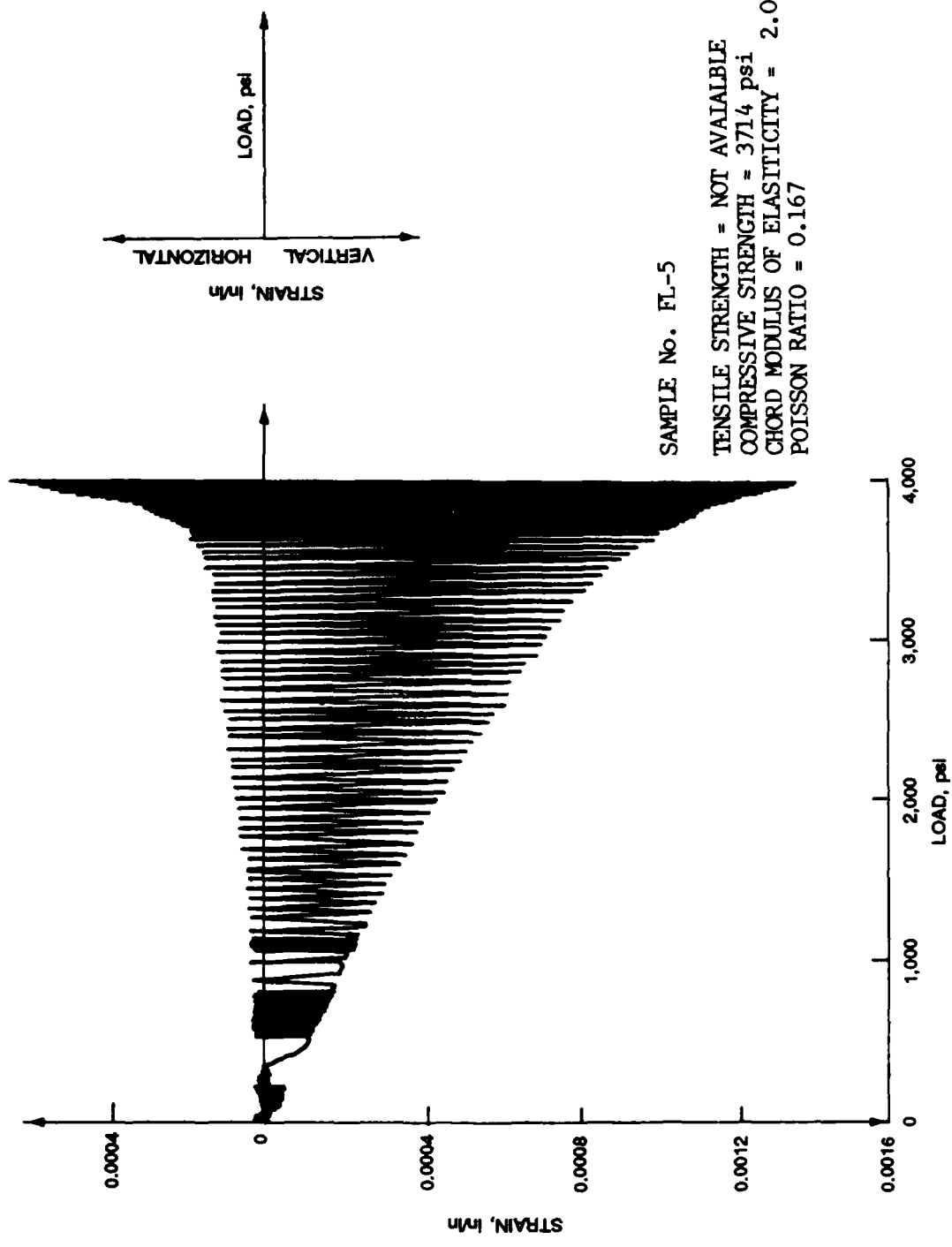
SAMPLE No. FL-5

TENSILE STRENGTH = NOT AVAIALBLE
COMPRESSIVE SIRENGTH = 3714 psi
CHORD MODULUS OF ELASTICITY =   2.063E + 06
POISSON RATIO = 0.167

Figure C.2 Concrete Test Data for Sample No. FL-5

LOAD, psi

STRAIN, in/in

VERTICAL   HORIZONTAL

5,000

4,000

3,000

LOAD, psi

2,000

1,000

0

0.0004

0

0.0004

0.0008

0.0012

STRAIN, in/in
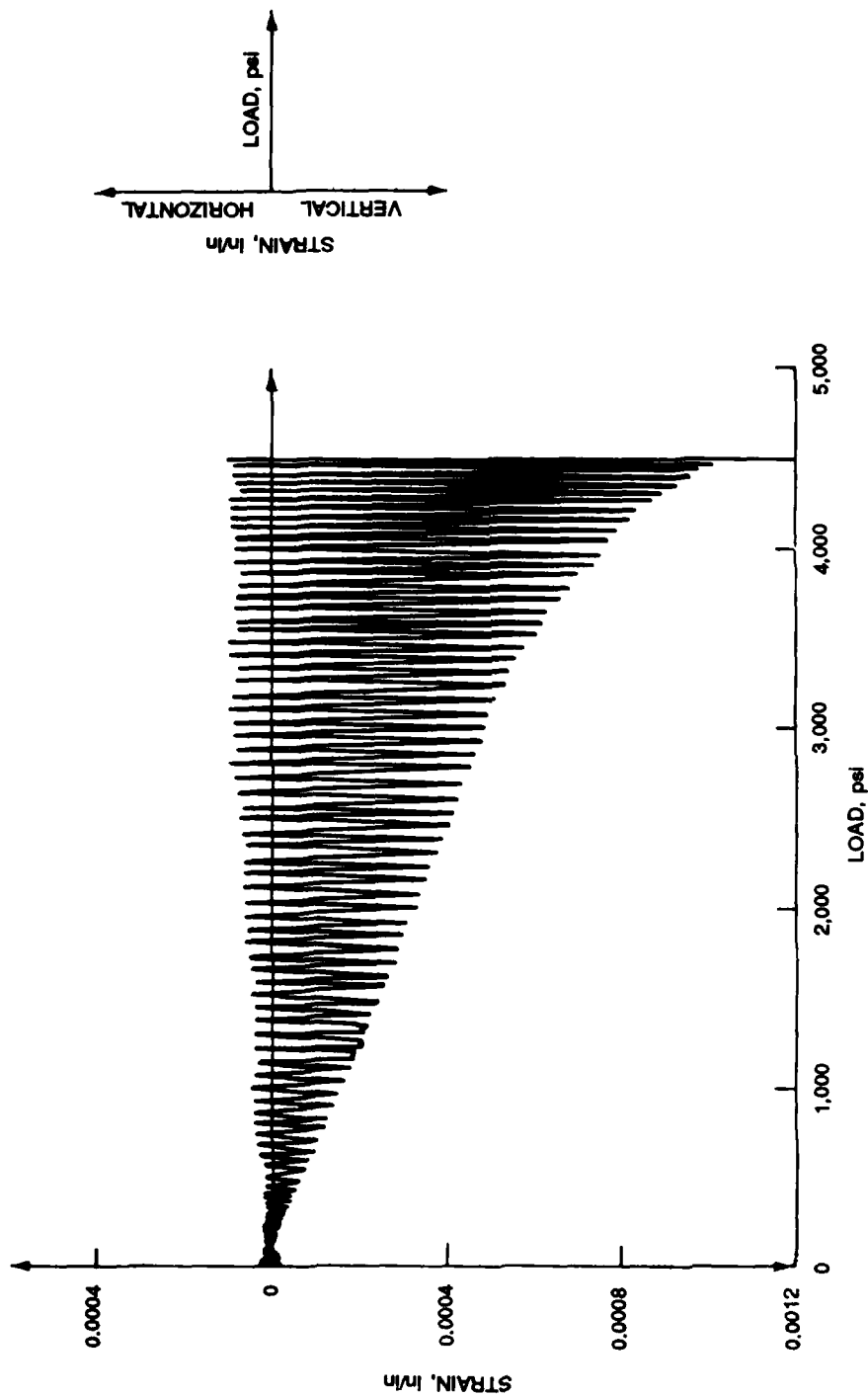
SAMPLE No. FL-6

TENSILE STRENGTH = NOT AVAILABLE
COMPRESSIVE STRENGTH = 4173 psi
CHORD MODULUS OF ELASTICITY = 2.319E + 06
POISSON RATIO = 0.167

Figure C.3 Concrete Test Data for Sample No. FL-6

SAMPLE No. FL-8

TENSILE STRENGTH = NOT AVAILBLE
COMPRESSIVE STRENGTH = 4598 psi
CHORD MODULUS OF ELASTICITY = 2.554E + 06
POISSON RATIO = 0.167
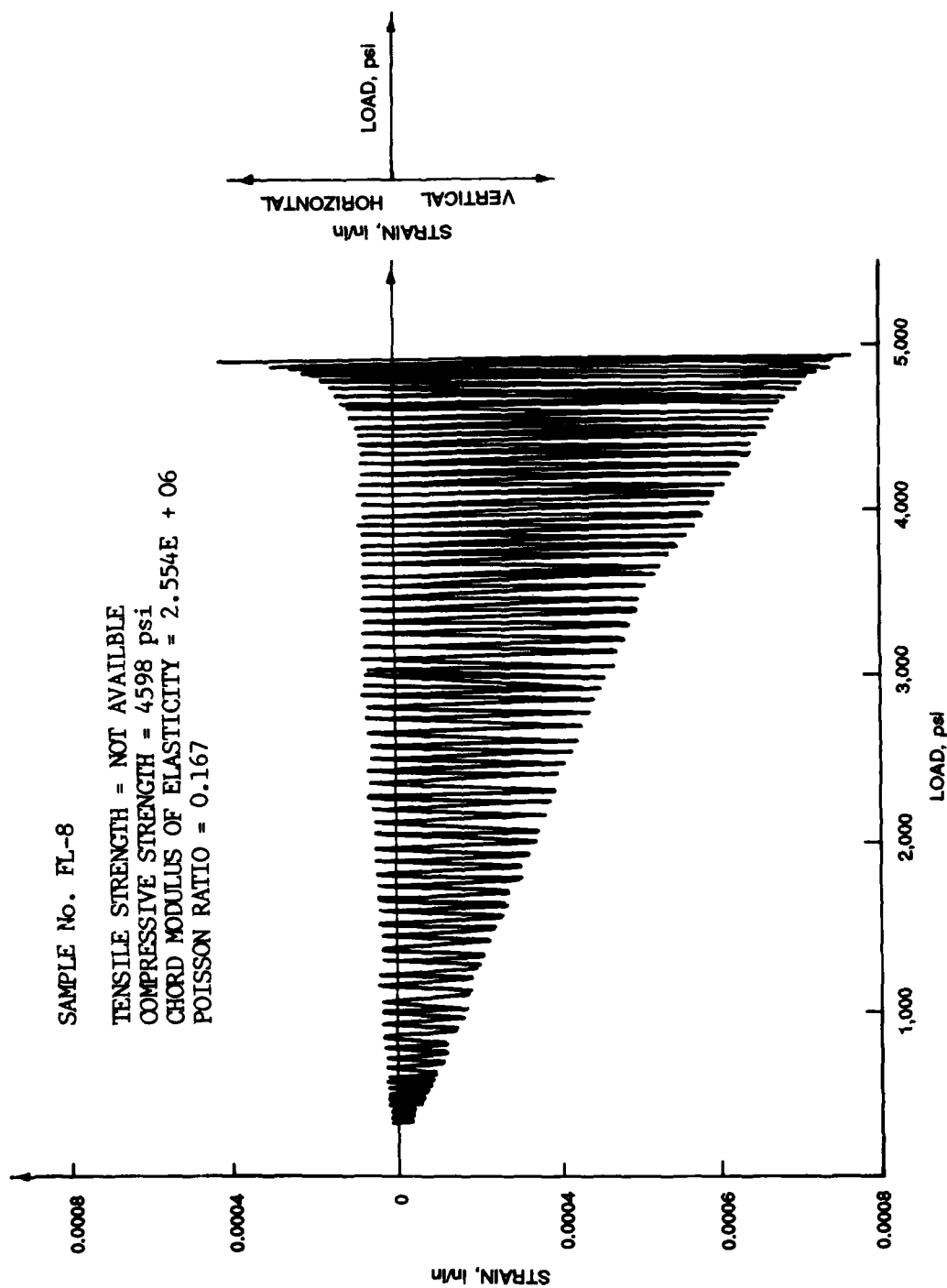
Figure C.4 Concrete Test Data for Sample No. FL-8
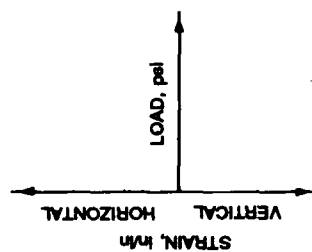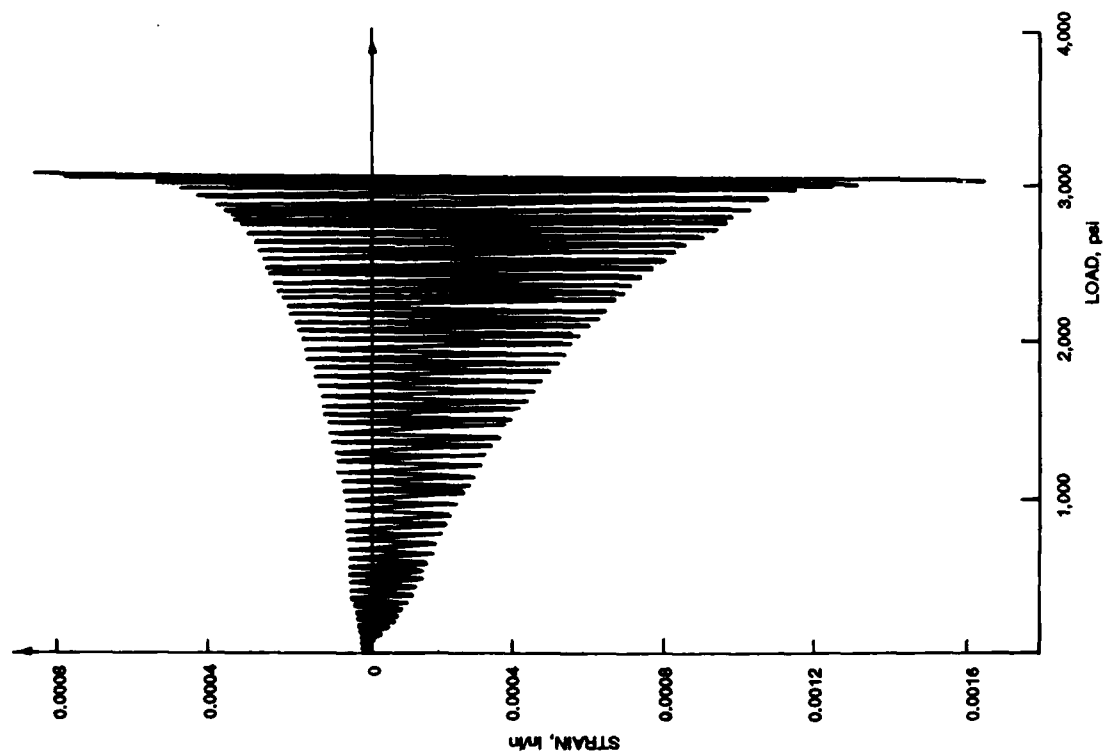
SAMPLE No. FL-9

TENSILE STRENGTH = NOT AVAIALBLE
COMPRESSIVE STRENGTH = 2829 psi
CHORD MODULUS OF ELASTICITY = 1.664E + 06
POISSON RATIO = 0.294

Figure C.5 Concrete Test Data for Sample No. FL-9
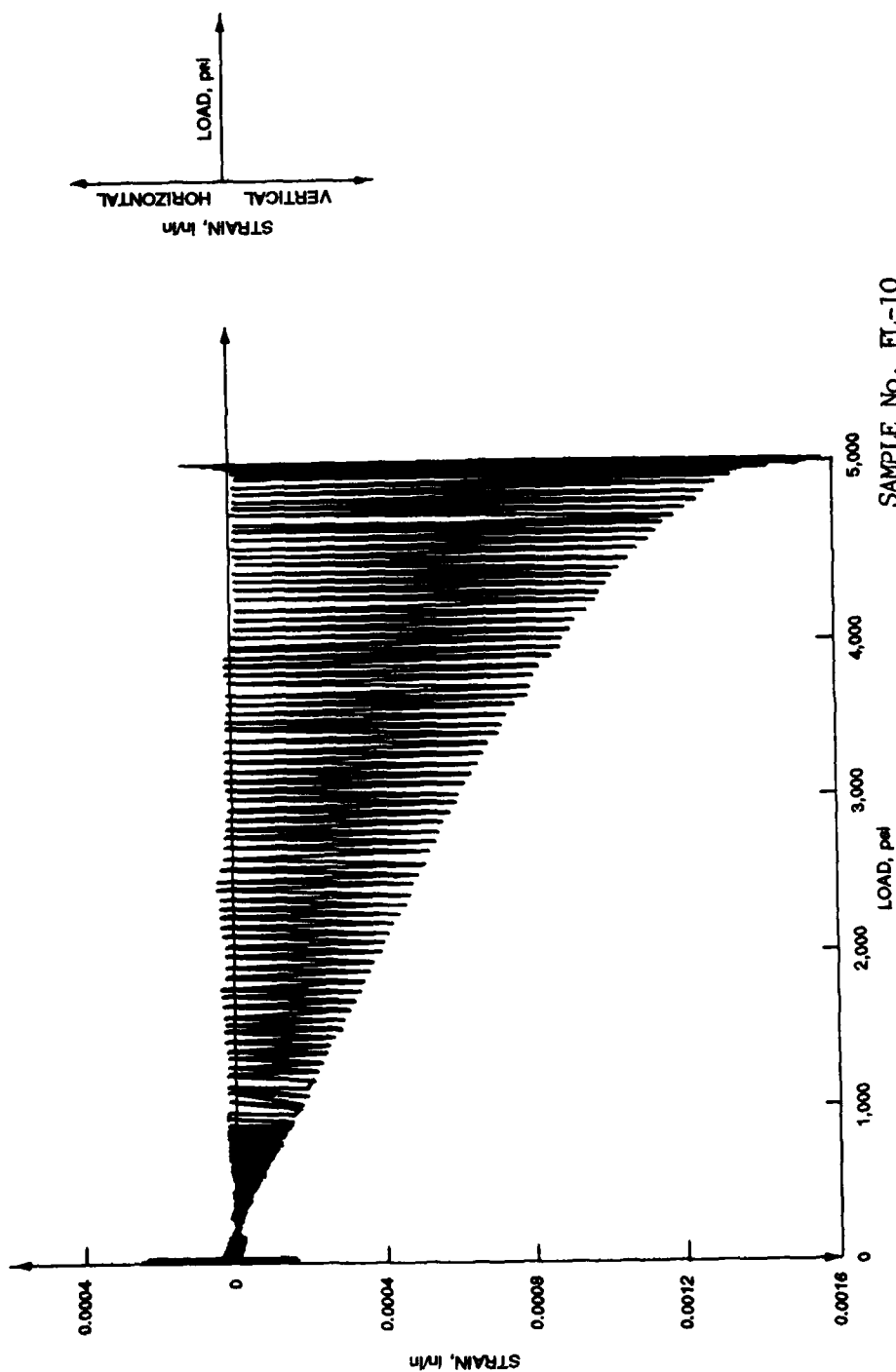
SAMPLE No. FL-10

TENSILE STRENGTH = NOT AVAILABLE
COMPRESSIVE STRENGTH = 4633 psi
CHORD MODULUS OF ELASTICITY = 1.930E + 06
POISSON RATIO = 0.083

Figure C.6 Concrete Test Data for Sample No. FL-10

LOAD, psi

VERTICAL HORIZONTAL

STRAIN, in/in

SAMPLE No. FL-11

TENSILE STRENGTH = NOT AVAILABLE
COMPRESSIVE STRENGTH = 4916 psi
CHORD MODULUS OF ELASTICITY = 1.891E + 06
POISSON RATIO = 0.173

1,000   2,000   3,000   4,000

LOAD, psi
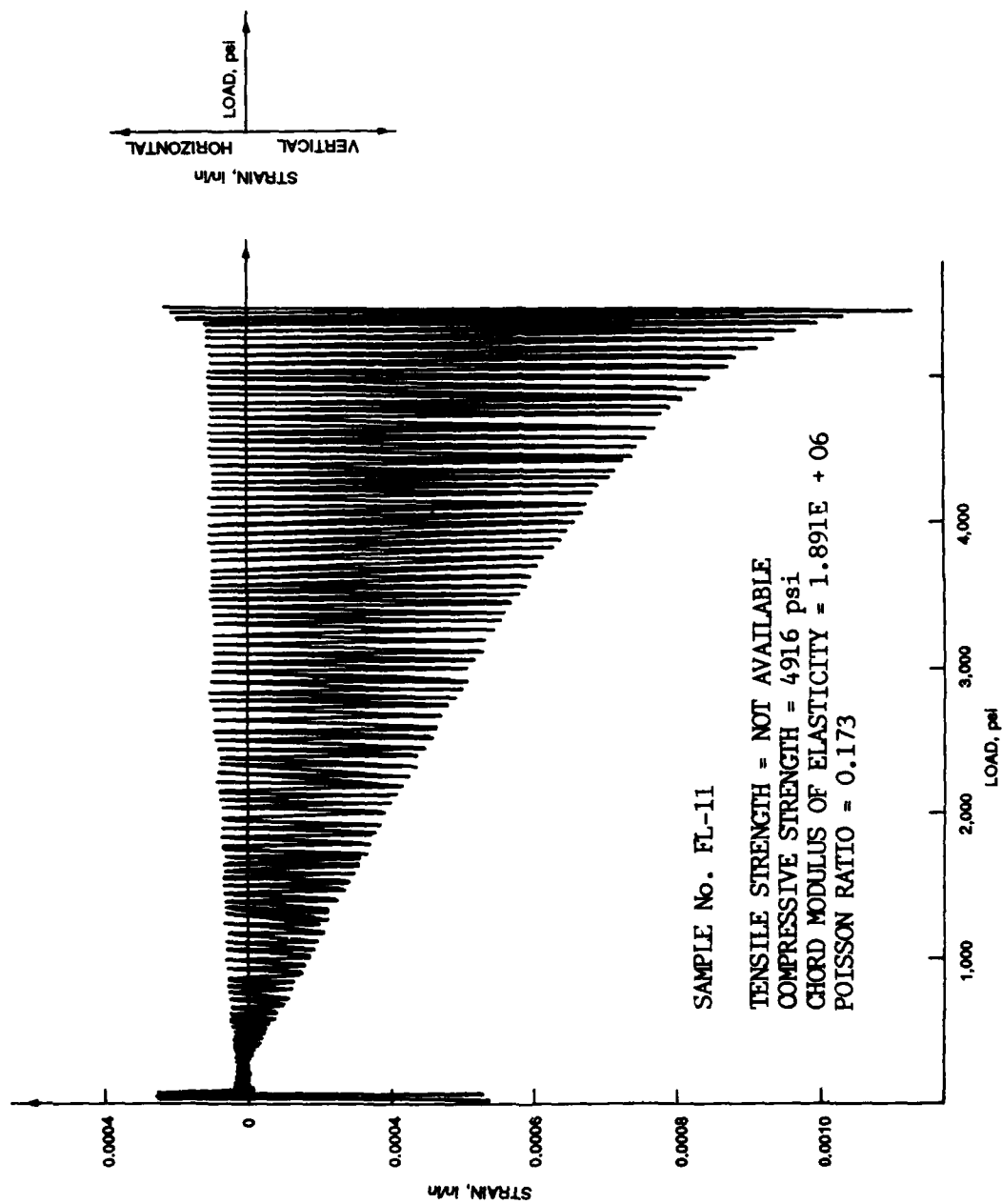
Figure C.7   Concrete Test Data for Sample No. FL-11

STRAIN, in/in

0.0004   0   0.0004   0.0006   0.0008   0.0010
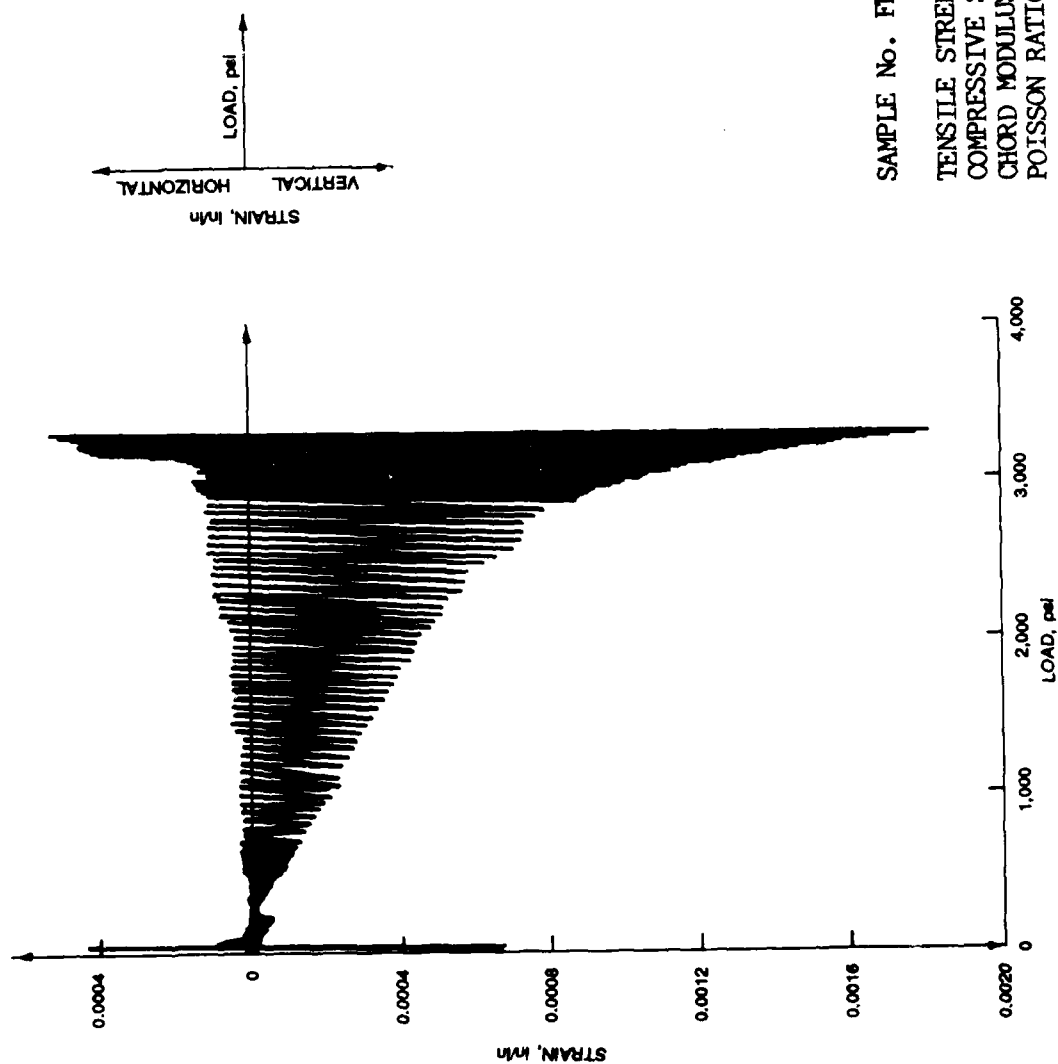
SAMPLE No. FL-12

TENSILE STRENGTH = NOT AVAILABLE
COMPRESSIVE STRENGTH = 2936 psi
CHORD MODULUS OF ELASTICITY = 1.677E + 06
POISSON RATIO = 0.171

Figure C.8 Concrete Test Data for Sample No. FL-12
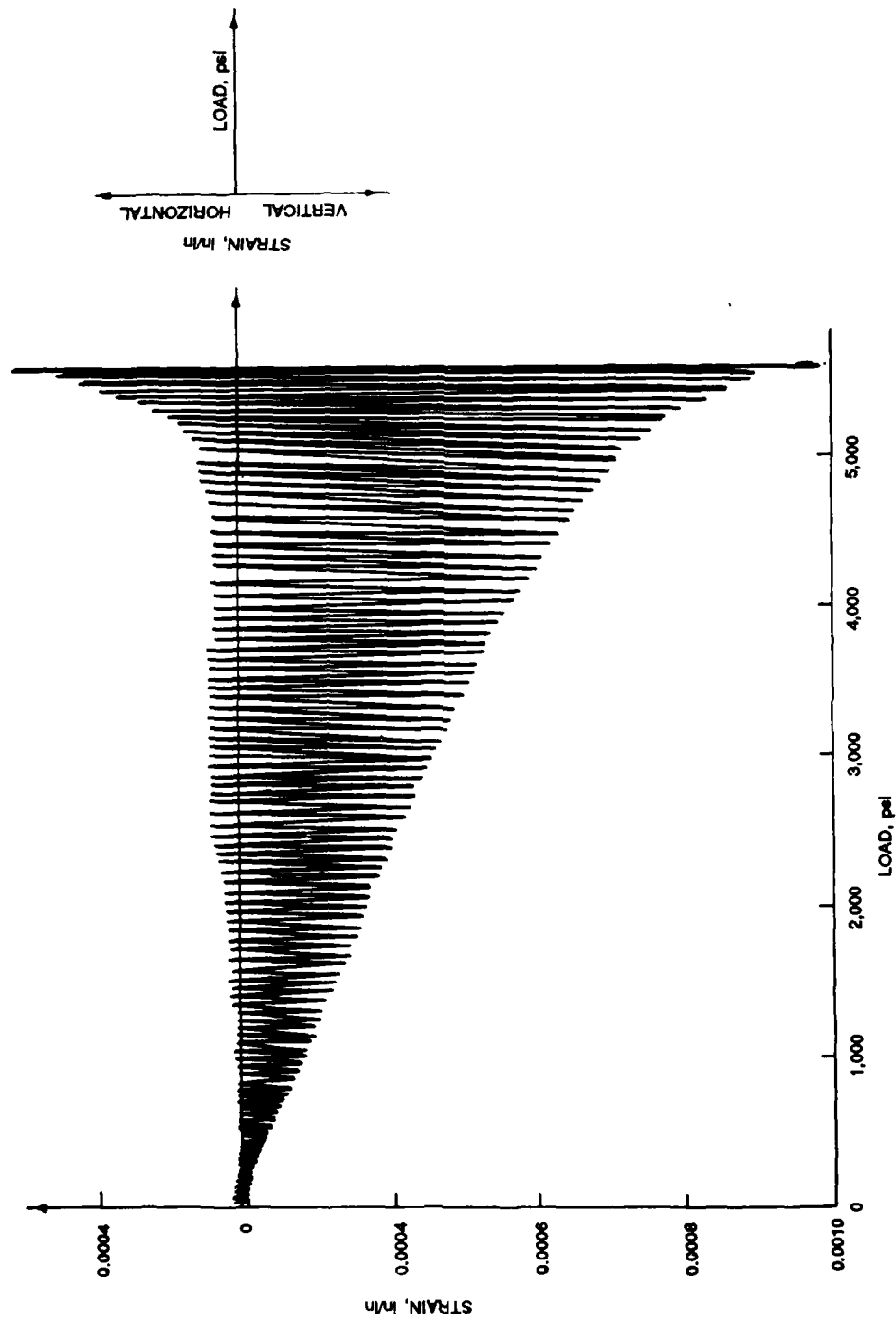
SAMPLE No. FL-15

TENSILE STRENGTH = NOT AVAILABLE
COMPRESSIVE STRENGTH = 4987 psi
CHORD MODULUS OF ELASTICITY = 2.168E + 06
POISSON RATIO = 0.130

Figure C.9   Concrete Test Data for Sample No. FL-15

SAMPLE No. FL-16

TENSILE STRENGTH = NOT AVAILABLE
COMPRESSIVE STRENGTH = 4173 psi
CHORD MODULUS OF ELASTICITY = 1.739E + 06
POISSON RATIO = 0.188
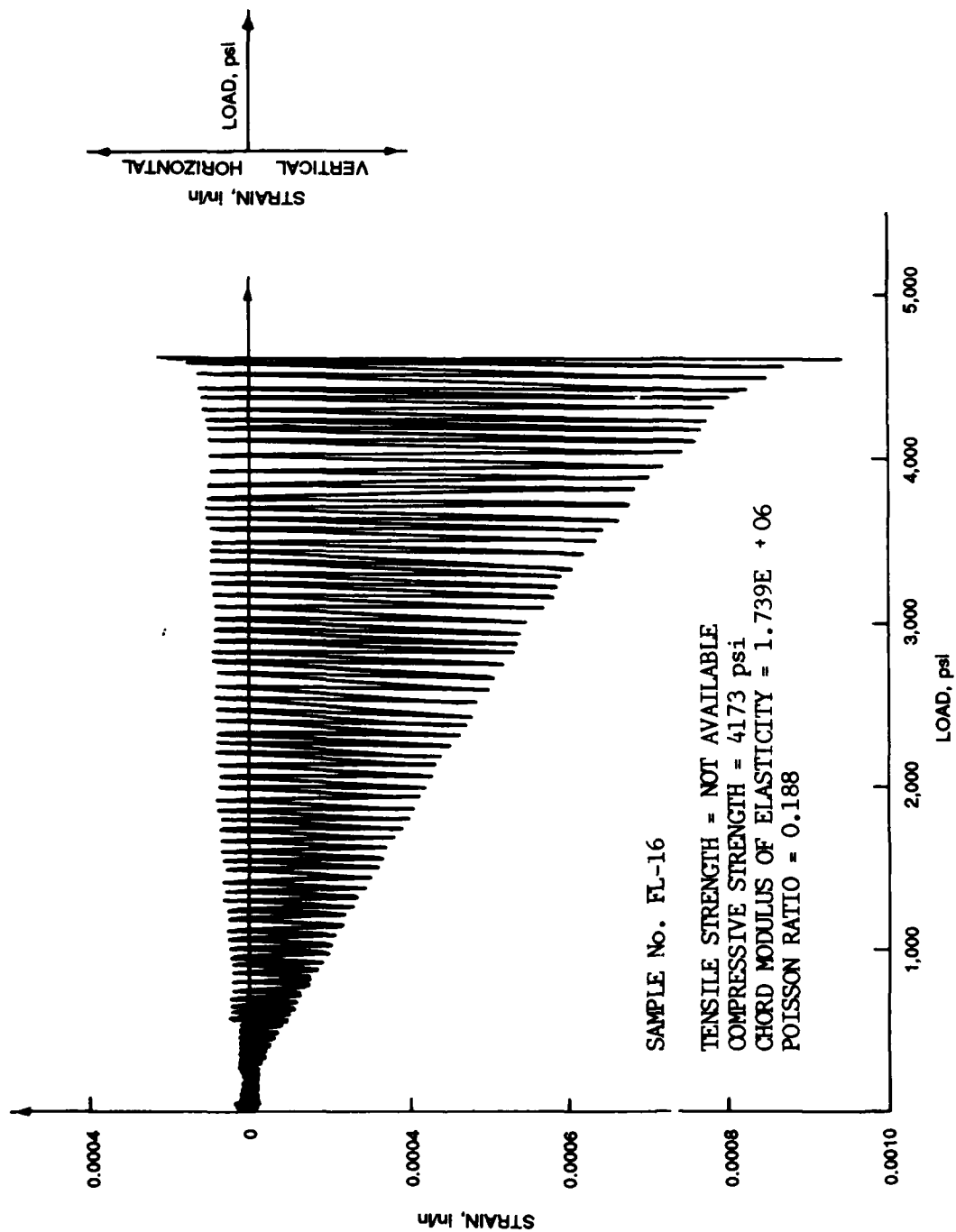
Figure C.10 Concrete Test Data For Sample No. FL-16

APPENDIX D

MASS CONCRETE PROPERTIES FOR SEISMIC EVALUATION OF

ENGELBRIGHT DAM
FOLSOM DAM
PINE FLAT DAM

JEROME M. RAPHAEL
Professor Emeritus
Consulting Civil Engineer
University of California
Berkeley, California

Report to
CORPS OF ENGINEERS
Department of the Army
Sacramento District
Sacramento, California

Under Contract No. DACW05-86-P-1049

October 1986
Revised January 1987

MASS CONCRETE PROPERTIES FOR SEISMIC EVALUATION OF

ENGELBRIGHT DAM

FOLSOM DAM

PINE FLAT DAM

Introduction

1. In connection with seismic stress analyses of Engelbright, Folsom,
and Pine Flat Dams, two independent sets of laboratory tests have been made of
elastic modulus and tensile strength of concrete cores taken from these dams,
with quite dissimilar results. It is the purpose of this report to analyze
and compare these test results, explain the discrepancies, and recommend prop-
erties to be used with the stress analyses in order to judge the seismic
safety of these dams.

Source of Test Data

2. Two independent sets of data were available for analysis. The first
set, which will be referred to as the USBR data, combine tests made by the
Corps of Engineers and the Bureau of Reclamation, and contained in two
reports:

> a. Mike Peabody and Fred Travers, "Testing of Engelbright Dam Cores
> under Rapid Loading Conditions," USBR Laboratory, Denver,
> October 1983.

> b. Mike Peabody and Fred Travers, "Testing of Folsom and Pine Flat
> Dam Cores under Rapid Loading Conditions," USBR Laboratory,
> Denver, December 1983.

3. The second set, referred to as the UCB data, is contained in a
single report entitled: Jerome M. Raphael, "Mass Concrete Tests for
Engelbright Dam, Folsom Dam, and Pine Flat Dam," Structural Engineering
Laboratory, University of California, Berkeley, September 1986.

4. The averaged results of tests for elastic modulus, Poisson's ratio
and splitting tensile strength under static and dynamic loadings are taken
from these reports and are summarized in Table D1.

5. No splitting tensile tests were reported by USBR/COE for Pine Flat
Dam. The cores tested by UCB for elastic modulus and Poisson's ratio for

D2

Table D1

Summary of USBR and UCB Test Results

Source	USBR	UCB
Engelbright Dam		
Chord modulus, static, psi $\times 10^6$	4.74	5.60
dynamic	4.63	5.79
Poisson's Ratio static	0.14	0.15
dynamic	0.21	0.18
Splitting tensile strength, static, psi	597.0	466.0
dynamic	585.0	624.0
Folsom Dam, lean concrete		
Chord modulus, static, psi $\times 10^6$	4.18	5.45
dynamic	4.50	5.95
Poisson's Ratio, static	0.14	0.18
dynamic	0.21	0.20
Splitting tensile strength, static, psi	482.0	363.0
dynamic	510.0	539.0
Folsom Dam, rich concrete		
Chord modulus, dynamic, psi $\times 10^6$	6.01	--
Poisson's Ratio, dynamic	0.22	--
Splitting tensile strength, static, psi	--	452.0
dynamic	655.0	649.0
Pine Flat Dam, lean concrete		
Chord modulus, static, psi $\times 10^6$	3.88	4.41
dynamic	3.43	4.42
Poisson's Ratio, static	0.15	0.24
dynamic	0.18	0.19
Splitting tensile strength, static, psi	462.0	377.0
dynamic	--	435.0
Pine Flat Dam, rich concrete		
Splitting tensile strength, static, psi	--	386.0
dynamic	--	526.0

Note: Static load test results reported by USBR were performed by COE.

Folsom and Pine Flat Dams were all taken from the "lean concrete" regions of
those dams.

6. On a first inspection of Table D1, some systematic discrepancies
can be seen. All the values of elastic modulus in the USBR column are lower
than all the values in the UCB column, averaging 81 percent of the UCB values.
For Poisson's ratio, the USBR values average slightly less than the UCB
values, but all seem credible. The USBR/COE tensile strength values are about
the same, whether tested under static or dynamic loadings, whereas the UCB
results all show a dynamic strength gain, averaging 35 percent.

7. To find reasons for these apparent discrepancies, it is necessary
to go back and examine the details of the tests themselves. Only then can we
present credible test results and give values to use with seismic analyses.

Test Cores

8. The cores used in the USBR/COE tests were 6-in. diameter. Follow-
ing my recommendations, the cores tested at UCB were 12-in. diameter. Fig-
ure C1 is a graphic representation of the data in Tables 3, 4, and 5 of the
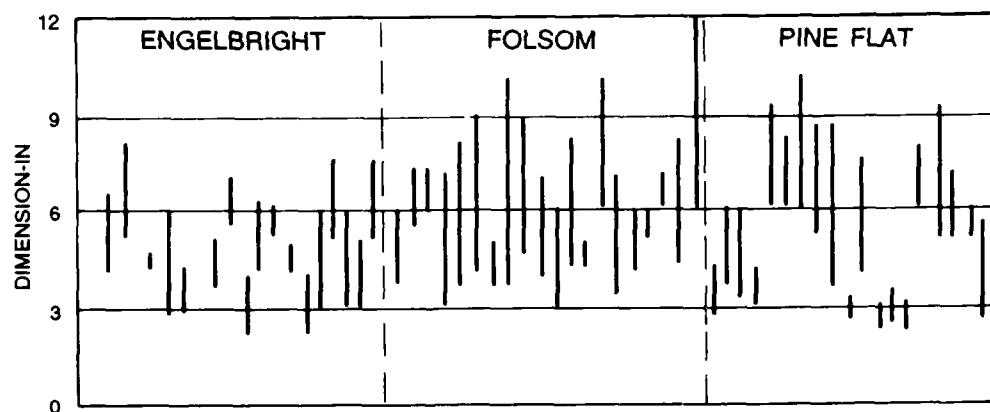
Figure D1. Length and width of largest aggregate

UCB report. Each line shows the length and width of the largest aggregate in
each of the cores tested for elastic properties. Percentages of cores with
particles larger than 6-in. were as follows: Engelbright - 50 percent,
Folsom - 90 percent, Pine Flat - 60 percent. In large aggregate concrete,
there is water gain during setting under each large particle, resulting in a
layer of significantly lower strength mortar, or laitance, at the interface.

If a single particle makes up most of the cross-section of a core, the apparent stiffness of the core will appear weaker. In some cases, the core may break during drilling at this interface, but in such cases that core would not be tested. UCB specified 12-in. cores for testing, assuming 6-in. MSA, with the expectation that the mortar around each large particle would pick up the load shed in the laitance layer. An additional factor is pertinent in judging the quality of the test results, the gage length of the transducers. The gage length of the USBR foil gages was 4-in. which is a fraction of the 6-in. MSA concrete. The gage length of the UCB compressometer was 12-in., twice the 6-in. MSA concrete. It is believed that measurements made with gages smaller than the MSA are unduly influenced by the deformation of a single large particle, rather than being responsive to the mass as a whole. For these reasons, any discussion of elastic properties will be based on the results of the UCB tests.

Modulus of Elasticity

9. The significant results of tests for elastic modulus have been assembled in Table D2. Previous tests of elastic moduli had showed increases of from 25 to 30 percent when comparing static and dynamic moduli, yet these tests show very little increase, the largest being only 9 percent. The Pine Flat core results were about equal in value. Suspecting that there might be some accident in the sampling that might account for this behavior, five Pine Flat cores that had been tested under static loads were retested under dynamic loads, only to find no changes in the earlier conclusion that there had been no increase in apparent stiffness in dynamic over static test results.

10. It is difficult to account for the differences between these results and those reported previously, but one fact stands out. The tests reported here are for concretes with 6-in. MSA; those reported previously were for concretes with 2.5- to 3-in. MSA. Perhaps the relatively greater amount of mortar in the earlier tests made the difference. It might be worthwhile to do further research along this line. In any case, the recommended elastic modulus for seismic loads is rounded off from the dynamic test results.

11. For slowly applied loads, such as dead load and reservoir water load, these loads are applied over periods of up to a couple of years, so slowly that creep enters into the calculations. When concrete is loaded, it

Table D2

Modulus of Elasticity

Dam	Engelbright	Folsom	Pine Flat
UCB Test results			
Static modulus	5.60	5.45	4.41
Dynamic modulus	5.79	5.95	4.42
Factor, E_d/E_s	1.03	1.09	1.002
Recommended values:			
Seismic loads	5.8	5.9	4.4
Dead load, water load	3.7	3.6	2.9

Note: All values of E in psi $\times$ 10^6.

deforms in proportion to the load, the factor in the elastic range being
termed the modulus of elasticity. If the load is sustained, the concrete
continues to deform, and this deformation, above the elastic deformation, is
termed creep. Thus for a load applied very slowly and sustained, such as dead
load and water load, the combination of elastic and creep deformations add
together to produce strain greater than that predicted by the elastic modulus.
This greater strain divided by the load gives what can be termed a sustained
modulus of elasticity, which depends on the characteristics of the particular
concrete, the age at which the concrete is first loaded, and the duration of
the load. It can readily be seen that to work this out for all parts of a dam
would be a next to impossible task. However, we do have a previous study to
serve as a guide. Creep and elastic deformations were tested for nine large
concrete dams, and the ratio of sustained modulus of elasticity to the
instantaneous modulus of elasticity determined for various ages at the time of
loading, and for a load duration of one year. The ratios averaged 60, 67, and
72 percent for loadings ages of 28, 90, and 365 days. An overall average of
67 percent of tested modulus is often used by designers for static loads, and
it has been used here, applying the factor 2/3 to the results of the static
tests.

Poisson's Ratio

12. Poisson's ratio has two distinguishing characteristics: (a) it is extremely difficult to determine experimentally with precision since it is the ratio of two very small quantities, and (b) once determined it has only minor effect on accepted analysis—a ten percent difference in Poisslon's ratio may affect the results of a stress analysis by about one percent. With these limitations in mind, the UCB test results are shown in Table D3. (The USBR results have been set aside since the 6-in. cores of concrete with predominantly larger than 6-in. MSA affects the results widely.)

Table D3
Poisson's Ratio

Dam	Engelbright	Folsom	Pine Flat
Test results from UCB report			
Poisson's ratio, static	0.15	0.18	0.24
Poisson's ratio, dynamic	0.18	0.20	0.19
Factor, μ_d/μ_s	1.20	1.11	0.79
Recomputed values			
Static	0.16	0.18	0.20
Dynamic	0.18	0.20	0.20
Recommended values			
Static and dynamic	0.17	0.19	0.20

13. All values reflect usual findings for Poisson's ratio, with the exception of 0.24 for the static tests of Pine Flat Dam concrete. In checking this, all plots of Poisson's ratio tests for all these dams were re-examined. In a number of plots, there seemed to be some slippage in the instrumentation, and the ratios were re-computed eliminating this source of error. The re-computed values are shown separately as the second group of data.

14. Finally, considering that small changes in Poisson's ratio have even smaller effect on stress analyses, and that the final stress analysis must add components of static and dynamic stress analyses, a single average

value of Poisson's ratio has been recommended for each dam, all as shown in Table D3.

Tensile Strength

15. Three methods are available for testing the tensile strength of concrete:

 a. Direct tensile test.

 b. Flexural test.

 c. Splitting tensile test.

16. Of the three, the splitting tensile test is the most accurate, and the most easily performed. It was the test used both in the USBR and the UCB tests. In a paper, "The Tensile Strength of Concrete," Raphael has described the comparative values of results of tests of concrete by the three methods, as well as relative values of strengths under static and dynamic testing conditions. It is shown in this paper that under dynamic loading, tensile strength is 50 percent higher than when tested under static loading.

17. With this in mind, it can be seen that in the USBR test results shown in Table D1, the tensile strengths are essentially equal for both static and dynamic tests. Furthermore, these tests were made with 6-in. cores, in which the grain boundaries of the 6-in. MSA concrete modified the test results unduly. For these reasons, the USBR split tension test results have not been considered in the recommended tensile strength for the three concrete dams.

18. The average results of split tensile tests of five types of concrete under static and dynamic tests have been assembled in Table D4. For Folsom and Pine Flat Dams, sets of 10 cores were taken from rich and lean concrete. It can be seen that the cores tested under dynamic conditions gave higher strength than those tested under static loads, by an average of 35 percent. The lowest increase, 15 percent was in the Pine Flat lean concrete set, which also gave the most widely dispersed test results, with a coefficient of variation of 26 percent.

19. The tensile test results shown in Table D4 cannot be used directly with stresses computed by finite element analyses to judge the relative safety of the three concrete dams, because the computed maximum tensile stresses have a built-in error, due to the shape of the stress-strain curve to failure of concrete. Finite element analysis is essentially a strain analysis—when the

Tensile Strength

Dam	Engelbright	Folsom		Pine Flat	
		Rich	Lean	Rich	Lean
UCB Test Results					
Split tension tests, static	466.0	452.0	363.0	386.0	377.0
Split tension tests, dynamic	624.0	649.0	539.0	526.0	435.0
Factor, f_{td}/f_{ts}	1.33	1.44	1.48	1.36	1.15
Apparent tensile strength					
$2.0\ (f_{ts})$	932.0	904.0	726.0	772.0	754.0
$1.30\ (f_{td})$	811.0	844.0	701.0	684.0	566.0
Recommended apparent					
Tensile strength	810.0	840.0	700.0	680.0	570.0

Note: All values in psi.

deformations and forces of all the finite elements are in balance, the strains everywhere are multiplied by the elastic modulus to give the stresses throughout the mass. Figure D2 shows the contradiction. For the strain at failure, the stress is actually that at Point B, whereas the straight-line stress prediction using a constant E will predict stress as shown by Point A. Thus to compare concrete strengths must be computed from the test strengths, using an appropriate multiplier. In the Raphael paper, two relationships can be derived:

a. Apparent dynamic tensile strength is twice the static tensile strength.

b. Apparent dynamic tensile strength is 1.30 times the dynamic tensile strength.

These relationships have been applied to the test results, as shown in Table D4. The lower of the two values found was then slightly rounded, and is the recommended apparent tensile strength for each concrete.

Conclusion

20. While we have a hundred years of experience with testing for elastic properties of mass concrete under normal loading conditions, it is only in the last fifteen years that we have been testing cores of mass concrete dams
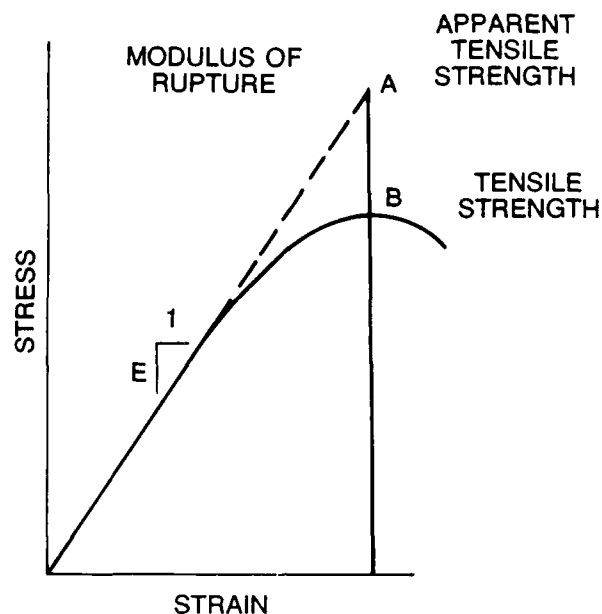
Figure D2. Typical stress-strain diagram for concrete

at speeds comparable to seismic loadings. It just happens that much of our experience in the past fifteen years has been gained on cores with aggregates smaller than 3-in., whereas the tests cited here have been made for the most part with concretes with 6-in. MSA. If these results differ slightly from those reported for other dams, the differences may lie in the actual combinations of mortar and largest aggregates. This is a point worthy of additional research, which lies beyond the scope of the present contract.

UCB Core Locations

21. Figures D3 through D6 show the drilling locations for the cores taken from Folsom Dam for the UCB tests.

D10

Sta 285-35.00

Sta 291-14.00

Sta 292-35.00 Flow

Sta 292-36.00

Top of na. El 848.0

Control line Sta 10+00.00

Slope 1 on 2

Retaining walls

Note
Compressor or generator
shall be
located here only,
except when drilling
crest holes

Slope 1 on 2

Penstock

Penstock

Penstock

Waterways

Tower

Gantry Crane

Flip Bucket

Toe of right
wing dam

Existing
excavation

Backfill with rock range

Pumping Station

Access
Road

0% Ramp

Power House

Slope
on 3.5

325

300

275

250

GRAPHIC SCALES

325

350

350

325

500

250

250

Notes

1 Contact USBR Project Engr for access road
to flip bucket area and access road to Adit
No 2

2 Concrete crest holes shall be drilled first.
Generator or compressor may be located on
crest of dam during this time. Generator
or compressor shall be located at Adit No 2
for all other holes

3 See Note 2, Page 3 — Same

4 See Note 3, Page 3 — Same

5 Each Drill Site Has One Or More Core Holes

Monoliths @ 50'0" 550'0

7 Monoliths @ 50'0

Tower

Adit No 1

Top of concrete : 484.00

42+50 Joints

Monolith joints

Adit No 2

Stairs

Adit No 3

IC-F-1

IC-F-2

IC-F-3

IC-F-4

DOWNSTREAM ELEVATION
SCALE 1"=50'-0"

DOWNSTREAM ELEVATION
SCALE

Notes
1 Stilling Basin is normally full of water

GRAPHIC SCALE

1"=50'-0"   0 25 50'   100'   150   200   250   300'

NOTES.

1. CONCRETE CORE HOLES ON DOWNSTREAM
   FACE OF DAM, ONLY, SHOWN ON THIS SHEET

2. CORE HOLES ON DOWNSTREAM FACE OF DAM
   MUST BE DRILLED HORIZONTALLY, NOT
   PERPENDICULAR TO FACE OF DAM.

3. CONCRETE CORE DRILL SITES

4. EACH DRILL SITE HAS ONE OR MORE CORE HOLES.

SECTIONAL ELEVATION

Domestic water available
in Control Tower

IC-F-8

Adit No 1   H P of roadway   El 480.50   El 435.62

El 444.0

El 416.0   Adit No 2   90° Float wall
                                 7'-0" ø Utility shaft
El 386.0   Escape shaft   Valve chamber   11'-6" x 12'-4" Elevator shaft
                                             IC-F-11
El 350.0   El 322.50

El 324.0   42" and 84" Dia pipes
                  El 305 @ Sta 0+00

                                             IC-F-12   Gate Chambers

6'-6" x 8'-3" Access gallery   Elevator stairs to
bottom of float well El 312.08

                                             4'-6" ø Sump

SECTIONAL PLAN AT EL 224.00

6'-6" x 8'-3" Inspection gallery

Control line Sta 10+00

11'-6" x 12'-4" Elevator shaft

7'-0" x 8'-3" Adit
Uniform slope, El 224.00
to El 222.82 @ face of dam

            12'-0'   7'-0" ø Utility shaft   10'-0" ø Hoist shaft
                     Grouting gallery
                     6'-6" x 8'-3" Inspection gallery
                     Gate Chambers
                                             Sump pump
                                             IC-F-14
                                             5'-0" x 7'-0"
                                             Inspection gallery

11'-10½"
Grouting gallery

6'-6" x 8'-3" Inspection gallery
10'-0" ø Hoist shaft
6'-6" x 11'-8" Inspection gallery
IC-F-12

SECTION A

SECTIONAL PLAN AT EL 322.50

Valve - 84" Pipe line   Valve - 42" Pipe line   12' wide pipe gallery
                             IC-F-11   7'-0" ø Utility shaft   10'-0" ø Hoist shaft   Control line Sta 10+00   5'-0" x 7'-0"
                                                                                                                                 Inspection gallery
Stair well to grouting gallery   2'-0" x 7'-0" Pipe gallery

Valve
Chamber   8" steel pipe   Float Well
             7'-0" x 7'-0" Adit   Stair well
                             11'-6" x 12'-4" Elevator shaft

Slope floor of 7'-0" x 7'-0"   Elevator & stair landing
Adit @ 0.0148 per foot toward   @ El 330.17
entrance   3'-0" x 8'-3" Gallery

Domestic water available in Control Tower

Piers

Spillway Crest El 418.00

IC-F-9   IC-F-10   Adit No 5

El 440.5
El 410.5
Escape shaft   El 380.5
El 350.5

5'-0"ø Hoist shaft

Gate chambers   Pipe gallery

5'-0" x 7'-0" Inspection gallery

5'-0" x 7'-0" Grouting gallery

IC-F-13

Adit No 4

IC-F-12

Gate chambers

5'-0" x 7'-0" Inspection gallery

IC-F-14

Sump

Ramps 8% max grade

Folsom Res
Water available
at Gate Chambers, typ

SECTIONAL ELEVATION

3'-6" x 3" inspection gallery
7'-0"ø Utility shaft

15'-0" 15'-0"

Grouting gallery   10'-0"

10'-0"ø Hoist shaft
6'-6" x 8" inspection gallery

Control line Sta 10+00

IC-F-12   5'-0" x 7'-0" Inspection gallery   IC-F-13

Gate chambers   gutter 4" deep
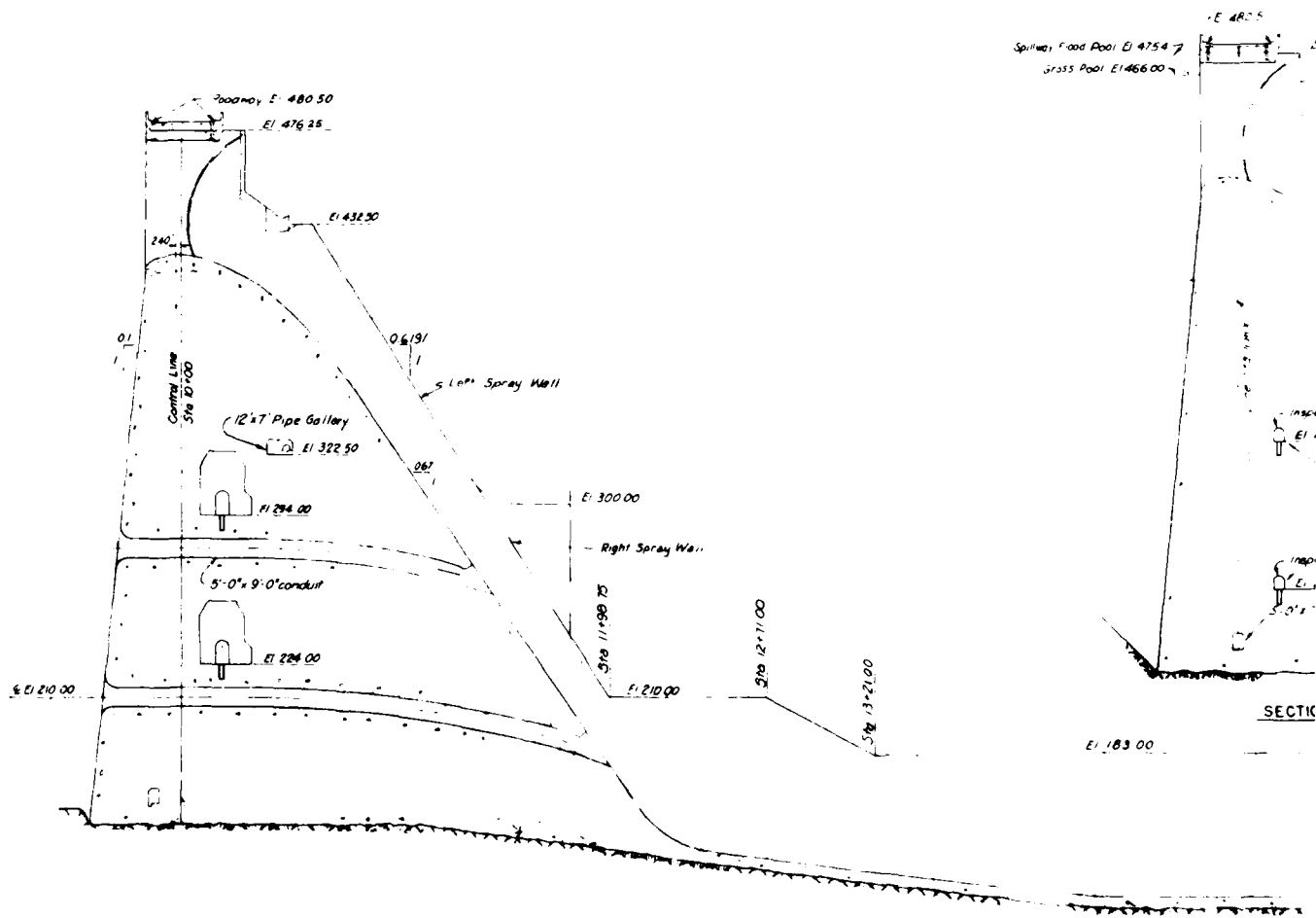
20   21   22

SECTIONAL PLAN AT EL 294.10

Notes

1 Core Holes are not to scale

2 Concrete core Drill Sites
   —◻ Interior or Crest Holes

3 Water pumped from small line from Folsom Lake
   may be used for drilling purposes.

4 Contractor shall verify and be responsible for
   compatibility between Government furnished
   power, water, and equipment and the Contractor's
   equipment.

5 ◻—● Interior Drill Sites

6 Each drill site has one or more holes.

5'-0" x 7'-0"
Inspection gallery
21

20   22

GRAPHIC SCALE

0   50   100   150   200   250   300

2

Poomoy El 480 50
El 476 25
El 432 50
240
Control Line Sta 10+00
0.6191
s Left Spray Wall
12'x7' Pipe Gallery
El 322 50
061
El 294 00
El 300 00
Right Spray Wall
5'-0"x 9'-0" conduit
El 224 00
El 210 00
El 210 00
Sta 11+98 75
Sta 12+11 00
Sta 13+21 00

Spillway Flood Pool El 475 4 7
Gross Pool El 466 00

El 480 5

El 183 00

SECTIO

SECTION THRU STILLING BASIN
SCALE 1" = 30'

1
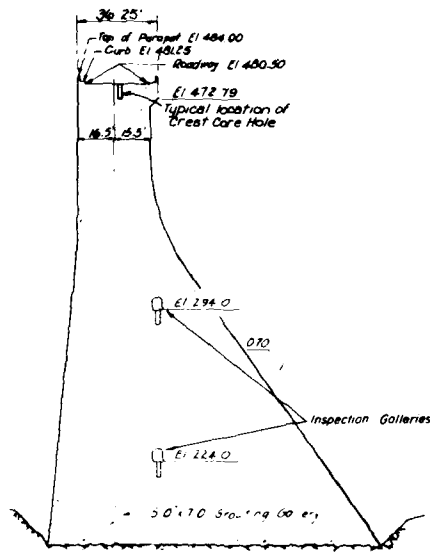
TYPICAL NON-OVERFLOW SECTION

STA 296+10   MONOLITH NO 22
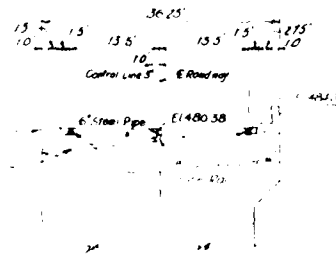
SCALE 1" = 50'

TYPICAL DETAIL AT TOP OF NON-OVERFLOW SECTION

SCALE 1" = 10'
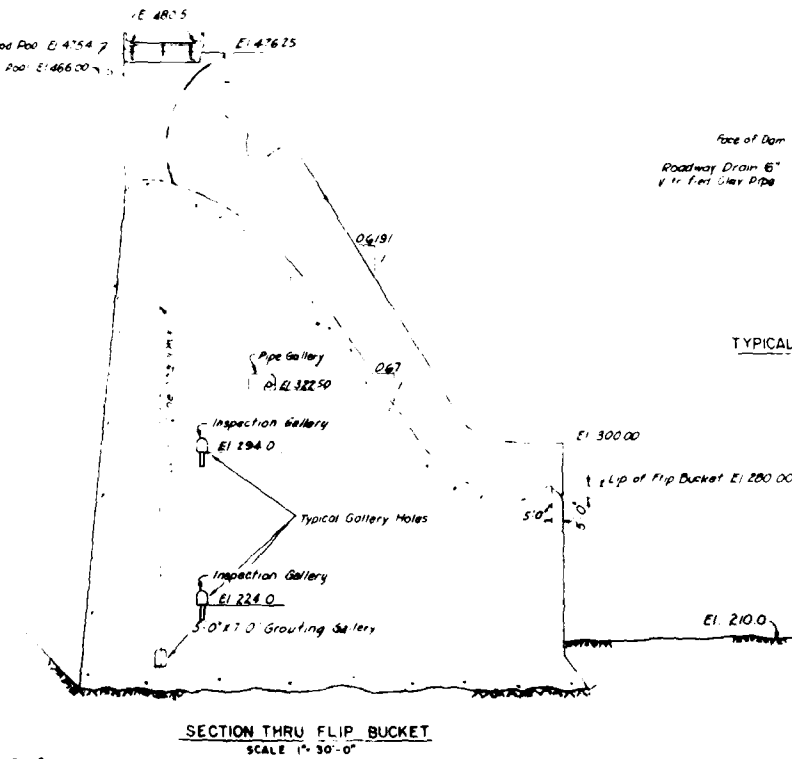
GRAPHIC SCALES

SECTION THRU FLIP BUCKET

SCALE 1" = 30'-0"

Notes

1 Core holes are not to scale

2 This sheet shows typical locations of core holes, not exact locations

APPENDIX E

ESTIMATES OF ROCK PROPERTIES FOR DYNAMIC ANALYSES OF
FOLSOM CONCRETE GRAVITY DAM

MEMORANDUM FOR RECORD

18 February 1988

Michael K. Sharp

MEMORANDUM FOR RECORD

SUBJECT: Estimates of Rock Properties for Dynamic Analyses of Folsom Concrete Gravity Dam

1. Background and Purpose. At the request of Dr. Robert L. Hall, Structures Laboratory, Geotechnical Laboratory personnel prepared estimates for several properties of the rock (granite or granodiorite) beneath the Concrete Gravity Dam at Folsom Dam, California. The properties included Young's modulus, compression-wave velocity, total unit weight, and Poisson's ratio and were to be representative values at a depth of 50+ ft. These values were provided for use in dynamic response analyses of the Concrete Gravity Dam.

2. Estimation Procedure. Initially, a figure was prepared to show the section under consideration (Figure 1). The Concrete Gravity Dam lies between Stations 285 + 35.00 and 299 + 35.00 and is bounded by the Right Wing Dam from Station 218 + 00.00 to 285 + 35.00 and the Left Wing Dam from Station 299 + 35.00 to 320 + 23.29. As shown in Figure 1, only the cross section from Station 265 + 00.00 to Station 310 + 00.00 is presented since it fully presents the data that are pertinent to the rock property estimates.

3. The compression-wave velocity for an elastic material can be expressed by the following equation given in Department of the Army, 1979.

$$V_p = \sqrt{\frac{E_d(1 - v)(144)}{p(1 - 2v)(1 + v)}} \qquad (1)$$

where V_p is the compression wave velocity in ft/sec, E_d is the dynamic Young's modulus in psi, p is the mass density which is equal to the bulk unit weight (γ in lb/ft^3) divided by g (32.2 ft/sec^2), v is Poisson's ratio and 144 converts units from in.^{2} to ft^2. To compute the dynamic Young's modulus, the above equation was rearranged to:

$$E_d = \frac{(V_p)^2(p)(1 - 2v)(1 + v)}{(1 - v)(144)} \qquad (2)$$

which gives an expression for the dynamic Young's modulus in terms of the other parameters. Procedures to determine and values established for each parameter for the rock beneath the Concrete Gravity Dam are discussed below. It was decided that since estimations were being made, lower, average, and upper bound values should be given.

4. Compression-Wave Velocity. Personnel of the Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory conducted extensive

SUBJECT: Estimates of Rock Properties for New Analyses of Folsom Concrete
Gravity Dam

geophysical investigations at the Folsom Project. These investigations
consisted of surface seismic refraction and crosshole seismic tests. A
description of the test and analysis procedures can be found in Department of
the Army, 1979. The locations and layouts for the tests are shown in Fig-
ure 1. One surface seismic refraction line, R1, was run on the downstream toe
of the Left Wing Dam as shown. The value 13,450 fps is a compression wave
velocity and is indicative of the material beginning at about Elevation
335 ft. The competent rock line, determined from boring data analyzed by the
Sacramento District, is seen to be fairly horizontal at this location at an
elevation of 295 ft. From surface seismic refraction theory, the depth of
investigation is approximately 1/4 to 1/3 the line length. Refraction line R1
was 400 ft long, which gives a depth of investigation of about 100-130 ft
below the ground surface. The ground surface along line R1 was between
Elevations 360 and 370 ft. Therefore, the 13,450 fps velocity should extend
to Elevations between 235 and 265 ft which is below the competent rock profile
line. Therefore, it is believed that the velocity of 13,450 fps is indicative
of very slightly weathered/fractured rock. Also shown at Station 303 + 00.00
are two boring sets (B1 and B2). Each set contained three holes (10 ft apart)
in which crosshole seismic tests were performed. The velocities ranged from
9,000 to 10,000 fps at the bottom of the holes. The slower velocities
obtained from the crosshole test are indicative of the more weathered rock at
this elevation. This procedure only tested the material located at a parti-
cular elevation between the three holes, one containing a seismic source and
the others containing receivers.

5. A second refraction line, R2, was run at the toe of the Right Wing Dam as
shown in Figure 1. Data from this line penetrated to a depth of about 85 to
110 ft and showed a velocity of 13,655 fps for the deepest layer. This layer
begins at Elevation 365 ft and extends to about Elevation 280 to 255 ft. This
velocity, which is almost the same as for line R1, is also believed indicative
of very slightly weathered/fractured granodiorite/granite. Crosshole set B3
is seen to penetrate about 15 ft into competent rock which is considerably
shallower than the 50+ ft criterion. The test results showed a velocity of
11,700 fps, at the bottom of the hole, which is probably a moderately
weathered/fractured granodiorite/granite. Therefore, based on the above
velocity information and examination of several references such as Department
of the Army, 1979, 14,000 fps was established as a lower bound velocity for
the competent rock beneath the Concrete Gravity Dam, 16,000 fps for an average
velocity and 18,000 fps as an upper bound.

6. <u>Bulk Unit Weight.</u> Very little data could be obtained on the properties of
the rock directly beneath the Concrete Gravity Dam. However, from six borings
drilled by Woodward-Clyde Consultants, 1983, some properties of the rock were
determined. A complete description of testing and results can be found in
their report. The six borings were labeled TH-A through TH-F, and their
locations are shown in Figure 1. From the boring data, rock type and descrip-
tion and the rock quality designation (RQD) were obtained. The rock was clas-
sified as granite or granodiorite. Borings TH-D through TH-F show very high
RQD's which indicates very sound rock with few fractures. The description of

the rock from these borings is similar to that of the borings on the Wing
Dams. From this, it was concluded that the rock beneath the Concrete Gravity
Dam and the rock beneath the Wing Dams were very similar if not the same.
Correlation of all this information allowed selections of lower, average and
upper bound bulk unit weights from granite/granodiorite values given by
Telford, 1976. These values were 167, 171, and 174 pcf, respectively.

7. **Poisson's Ratio.** Since values for the velocity and total unit weight
had been estimated as outlined above, only Poisson's ratio was left to be
obtained. Usually, Poisson's ratio is determined with seismic data from a
knowledge of the shear wave velocity and P-wave velocity of the in situ
material. Since only the P-wave velocity and no shear wave velocity were
available, this usual procedure could not be employed. Although shear wave
velocities were obtained from the crosshole seismic tests, the holes did not
penetrate deeply enough into competent rock to provide the shear wave velocity
in the zone of interest. Estimates of Poisson's ratio were determined by
senior geophysicists, EEGD, based on values presented by Lama and Vutukuri,
1978. As with the velocity and total unit weight estimates, lower, average,
and upper bound values of Poisson's ratio were estimated. The recommended
values are 0.20, 0.25, and 0.30, respectively.

8. **Dynamic Young's Modulus.** The lower, average, and upper bound values of
compression-wave velocity, bulk modulus and Poisson's ratio described in the
preceeding paragraphs were used in the equation for Young's modulus (see
Equation 2) to compute the lower, average, and upper bound modulus values.

The calculations resulted in dynamic Young's moduli of 5.8×10^6, 7.9×10^6,
and 11.0×10^6 psi for the lower, average, and upper bounds, respectively.
These results along with the values of all parameters used in the calculations
are presented in Table 1.

Encls   MICHAEL K. SHARP
                                         Earthquake Engineering and
                                         Geophysics Division

TABLE 1

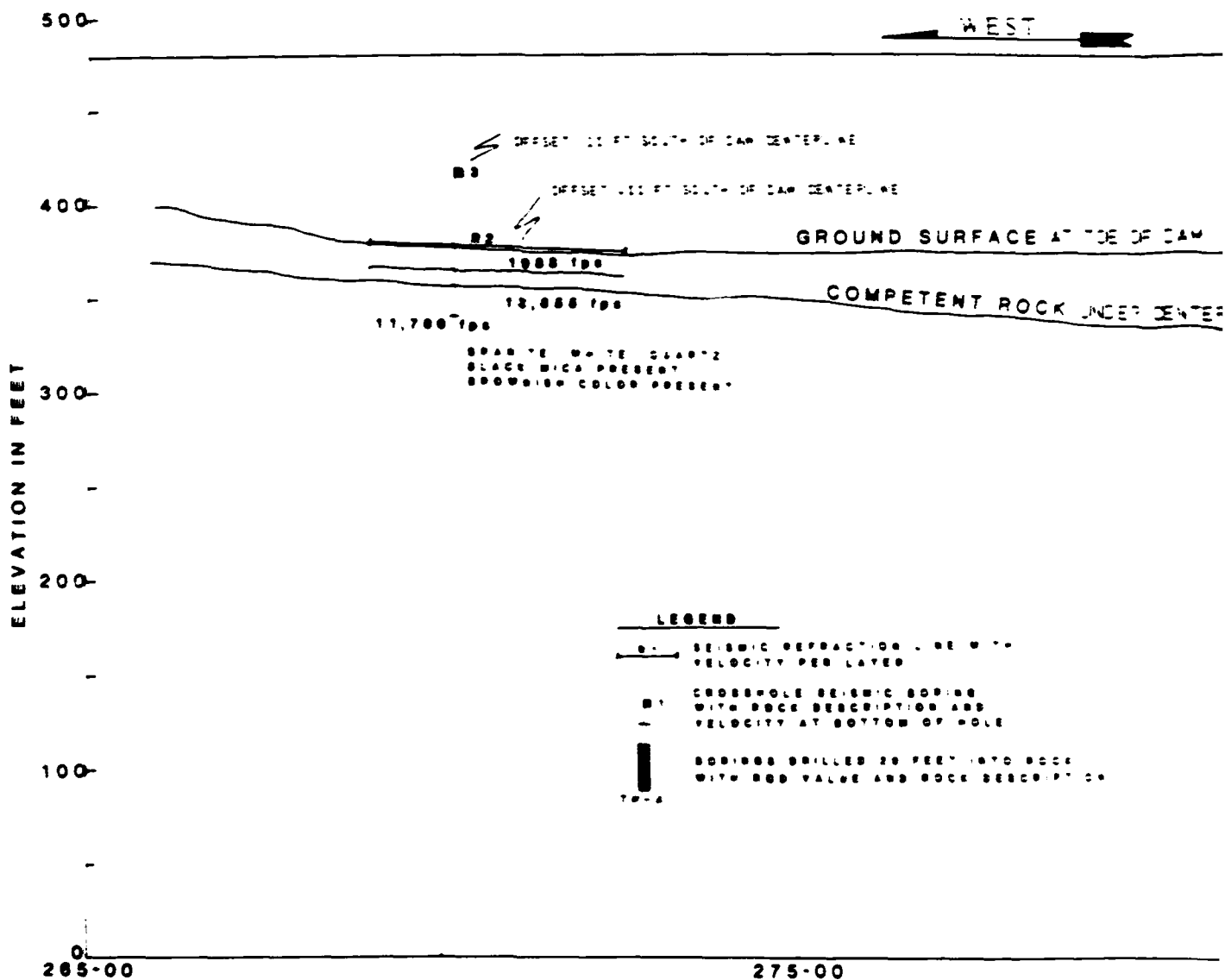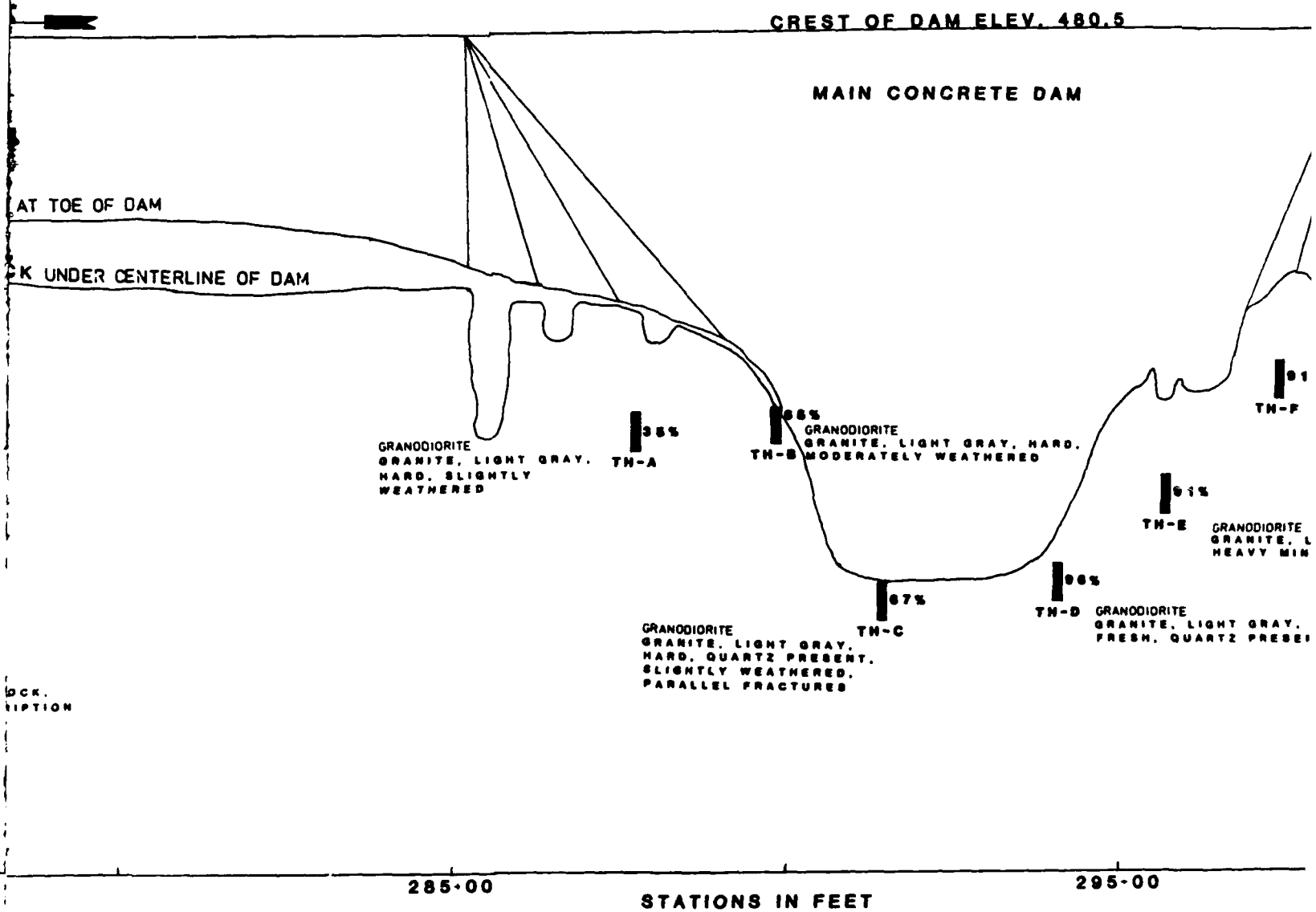| Measured Compression-Wave Velocity V_p (fps)	Assumed Poisson's Ratio v	Assumed Bulk Unit Weight γ pcf	Calculated Young's Modulus E_d psi	Remarks
14,000	0.30	167	5.8×10^6	Lower Bound
16,000	0.25	171	7.9×10^6	Average
18,000	0.20	174	11.0×10^6	Upper Bound

REFERENCES

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Telford, W. M., Geldart, L. P., Sheriff, R. E., and Keys, D. A. 1976. "Applied Geophysics," Cambridge University Press, New York, N.Y.

Lama, R. D., and Vutukuri, V. S. 1978. "Handbook on Mechanical Properties of Rocks, Volume II," Trans Tech Publications, Clausthal, Germany.

Woodward-Clyde Consultants. 1983. "Investigation of Concrete and Foundation Conditions at Folsom Dam," Woodward-Clyde Consultants, Walnut Creek, California.

WEST

DFFSET 100 FT SOUTH OF DAM CENTERLINE

B 3

DFFSET 100 FT SOUTH OF DAM CENTERLINE

B 2

GROUND SURFACE AT TOE OF DAM

1988 fps

COMPETENT ROCK UNDER CENTER

13,888 fps

11,700 fps

GRANITE WHITE QUARTZ
BLACK MICA PRESENT
BROWISH COLOR PRESENT

LEGEND

SEISMIC REFRACTION LINE WITH
VELOCITY PER LAYER

CROSSHOLE SEISMIC BORING
WITH ROCK DESCRIPTION AND
VELOCITY AT BOTTOM OF HOLE

BORINGS DRILLED 20 FEET INTO ROCK
WITH RQD VALUE AND ROCK DESCRIPTION

TW-4

ELEVATION IN FEET

500
400
300
200
100
0

265-00   275-00

CREST OF DAM ELEV. 480.5

MAIN CONCRETE DAM

AT TOE OF DAM

K UNDER CENTERLINE OF DAM

35%
TH-A

68%
TH-B

GRANODIORITE
GRANITE, LIGHT GRAY, HARD,
MODERATELY WEATHERED

GRANODIORITE
GRANITE, LIGHT GRAY,
HARD, SLIGHTLY
WEATHERED

67%
TH-C

GRANODIORITE
GRANITE, LIGHT GRAY,
HARD, QUARTZ PRESENT,
SLIGHTLY WEATHERED,
PARALLEL FRACTURES

98%
TH-D

GRANODIORITE
GRANITE, LIGHT GRAY,
FRESH, QUARTZ PRESEI

91
TH-F

91%
TH-E

GRANODIORITE
GRANITE, L
HEAVY MIN

OCK.
IPTION

285·00   295·00
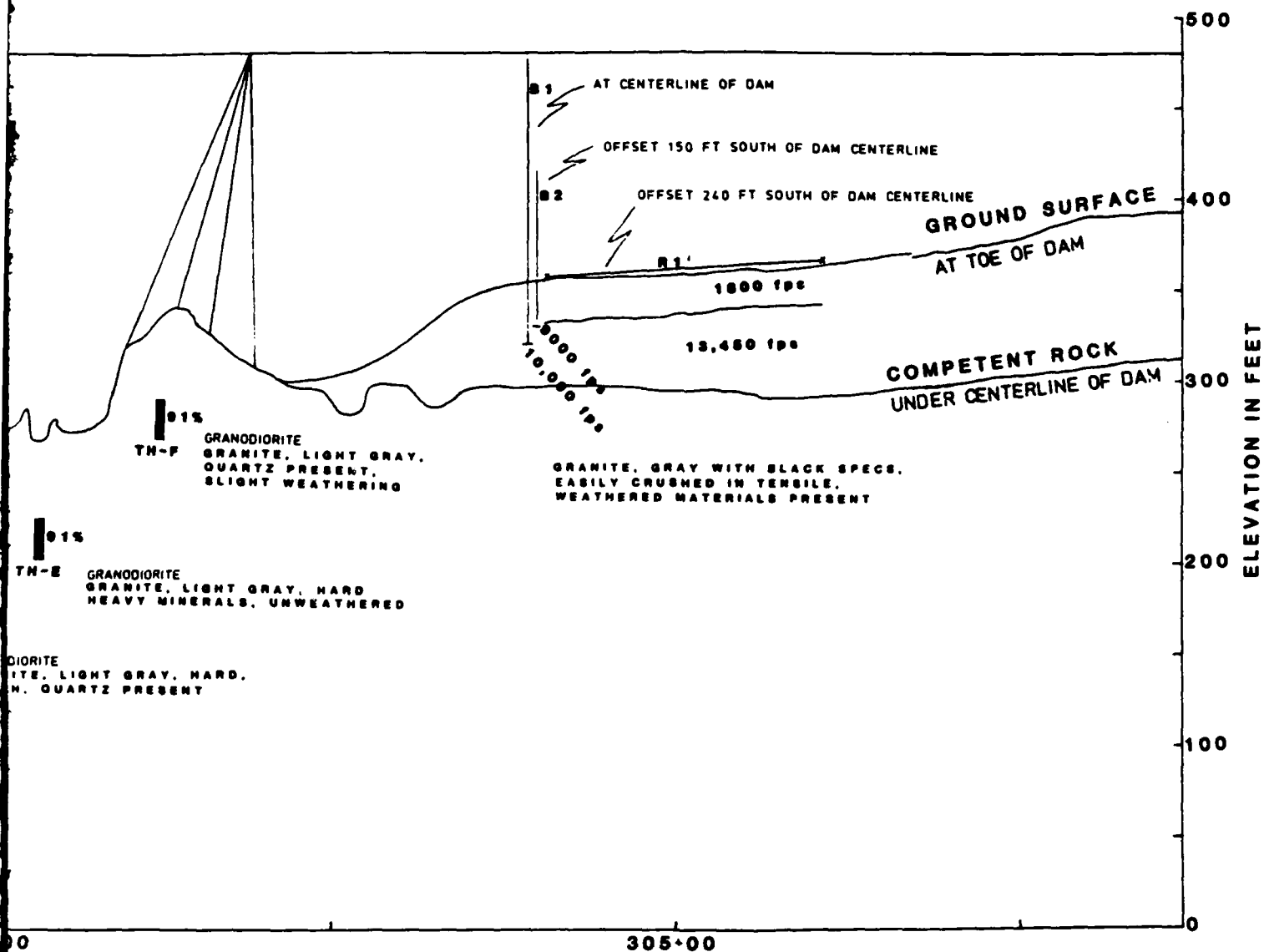
STATIONS IN FEET

2

FIGURE 1. CROSS SECTION OF CONCRETE GRAVITY DAM

APPENDIX F

SELECTION OF CRITICAL SECTION

APPENDIX F: SELECTION OF CRITICAL SECTION

1. At the outset of these analyses, the tallest nonoverflow monolith was assumed to be the critical section. This assumption was made since the tallest was the most flexible and therefore was assumed to have the highest dynamic stresses. This assumption was verified by analyzing four additional typical cross sections. The sections were the shortest nonoverflow monolith, a spillway monolith with a pier, and a spillway monolith without a pier.

2. The identical material properties were used for all analyses. The concrete modulus of elasticity used in all analyses was 5.8×10^6 kip/ft^2. This value corresponds to the value obtained from rapid load tests as reported in Part III of this report. From the information provided by the U.S. Army Engineer District, Sacramento, the unit weight of the concrete dam was 158 pcf. A value of 11×10^6 psi was used for the foundation modulus. A constant hysteretic damping factor $S = 0.1$ for the concrete dam and $F = 0.1$ for the foundation rock were used. The absorptive nature of the reservoir bottom was characterized by assuming a conservative wave reflection coefficient of 0.99.

3. The horizontal accelerogram, designated EQ2, was selected as the earthquake record for these analyses. The peak accelerogram of these records was 0.35 g. The vertical accelerogram was generated from the horizontal component by increasing the frequency content by 1.5 and by multiplying the amplitudes -0.6. The parameters which control the response computation in the program EAGD-84 were the same as those in Part IV (Response Parameters) of this report.

4. The shortest nonoverflow monolith was idealized as an assemblage of 200 planar, four mode, finite elements as shown in Figure F1. The maximum stress for the earthquake designated EQ2, -HV was 584 psi at elevation 196 on the downstream side. The time of the maximum stress was 2.98 sec. This value is less than 916 psi for the same earthquake loading acting on the tallest nonoverflow monolith. This indicates that the tallest nonoverflow monolith is the most critical section.

5. A typical spillway section without a pier was idealized with an assemblage of 148 planar, four node, finite elements as shown in Figure F2. Two different models were analyzed. The first model considered the structure

F2

to be completely solid, thus ignoring the voids created by the two 5-ft by 9-ft conduits and three machine rooms as shown in Figure F3. The maximum tensile stress of 101 psi for this analysis occured at elevation 272 ft on the downstream face of the dam at a time of 1.15 sec. In a second model, the material properties of sections A and B, shown in Figure F4, were modified to account for the conduits. The modulus of elasticity and the mass at these sections was reduced by a factor equal to the ratio of the total volume of the element one monolith in depth to the volume of the conduit. The maximum tensile stress of 101 psi for this analysis occured at elevation 296 on the downstream face of the dam at a time of 2.99 sec.

6. A spillway section with a pier was modelled with 228 four node planar elements as shown in Figure F5. The pier's mass and modulus of elasticity was reduced by a factor equal to the ratio of the monolith width to the pier width. The conduits were also modeled in this analysis by reducing the mass and modulus of elasticity of elements corresponding to the location of the conduits. This model is only valid for calculating the stresses in the mass concrete. A three-dimensional (3-D) model would be required to calculate stress in the reinforced piers. This analysis resulted in a maximum tensile stress of 311 psi at elevation 368 on the downstream face of the monolith. This maximum tensile stress occured at 2.8 sec. The maximum compression stress was 813.5 psi.

7. These analyses demonstrate that the tallest nonoverflow monolith is the critical section; however, the tallest monolith with a tower needs to be analyzed separately in order to determine the effect the tower has on the dynamic response of this section. The analysis of the nonoverflow monolith with the tower is a 3-D problem. The tower can be modeled as an equivalent two-dimensional (2-D) plane stress model by adjusting the mass and the modulus of elasticity of different elements until the natural frequency of the 2-D model matches the natural frequency of the 3-D tower model. Such an analysis by the Sacramento District for Pine Flat Dam resulted in the equivalent 2-D system with seven different materials with their properties as shown in Figure F6. This model approximately represents the mass and stiffness of the 3-D tower but cannot be used to calculate the stresses in the tower section. A 3-D analysis of the entire tower monolith would be necessary to calculate stresses in the tower. The 2-D tower model was developed by the Sacramento District for the Pine Flat Dam. The tower on the Pine Flat Dam is 33 ft along

the axis of the dam and 27 ft normal to the dam and extends 37.5 ft above the top of the section. The tower on monolith 11 of Folsom Dam is 32.5 ft along the axis of the dam and 26 ft normal to the dam and extends 30 ft above the top of the section. Since the tower on the Folsom Dam is very similar to the Pine Flat tower, the tower model developed by the Sacramento District (Figure F6) for the Pine Flat Dam was placed on the finite element model of the tallest monolith (Figure F7). This model includes 411 nodes and 346 planar four node elements.

8. The mode shapes and natural frequencies of the nonoverflow monolith are changed only slightly with the addition of the tower. Table F1 lists the changes in the natural frequencies for the first four mode shapes. The first four mode shapes of the tallest nonoverflow monolith were changed very little by the addition of the tower, as seen in Figures F8-F11.

9. The horizontal accelerogram labeled EQ-2 in Part 4 of the main text of this report was used for these comparisons. The direction of -HV was to produce the highest stresses, as seen in Table 5. The intermediate foundation modulus of 7.9×10^6 psi, with a corresponding $\alpha = 0.79$, was used for the seismic analysis. The response parameters presented in Part 4 were used for this analysis.

10. Figure F12 displays the maximum principal stresses that occurred during the entire earthquake time-history along the upstream face with the tower and without the tower. Figure F13 displays the same stresses along the downstream face. The element principal stress time-history for the three elements with the highest stresses is shown in Figures F14 through F16. These plots indicate that the maximum repeatable tensile stress is 290 psi, and the maximum compressive stress is 921 psi. The extent of tensile stresses is shown in Figure F17, which displays the envelope of the maximum principal stresses. Contours are not shown for the tower, since this analysis is only appropriate for determining stresses in the mass concrete dam section. These results indicate that the addition of the tower to the tallest nonoverflow monolith did not significantly affect its seismic response. The analysis did not evaluate the stresses in the tower. Since a failure of tower is not critical for structural stability of the dam or the operation of the dam, a detailed structural seismic analysis of the tower was not performed.

11. The above analyses demonstrate that the critical monolith for evaluating maximum tensile stresses is the tallest nonoverflow monolith. The

spillway monoliths are more massive and less flexible than the critical section and, therefore, have lower maximum tensile stresses. The increase in the dam due to the presence of the tower (Figures F12 and F13) are not large enough to change the overall conclusion of this report regarding safety of the dam.

Table F1

Natural Frequencies

Mode Number	Nonoverflow Monolith	Nonoverflow Monolith with Tower
1	4.6 Hz	4.5 Hz
2	9.5 Hz	9.8 Hz
3	9.5 Hz	10.0 Hz
4	16.7 Hz	17.2 Hz
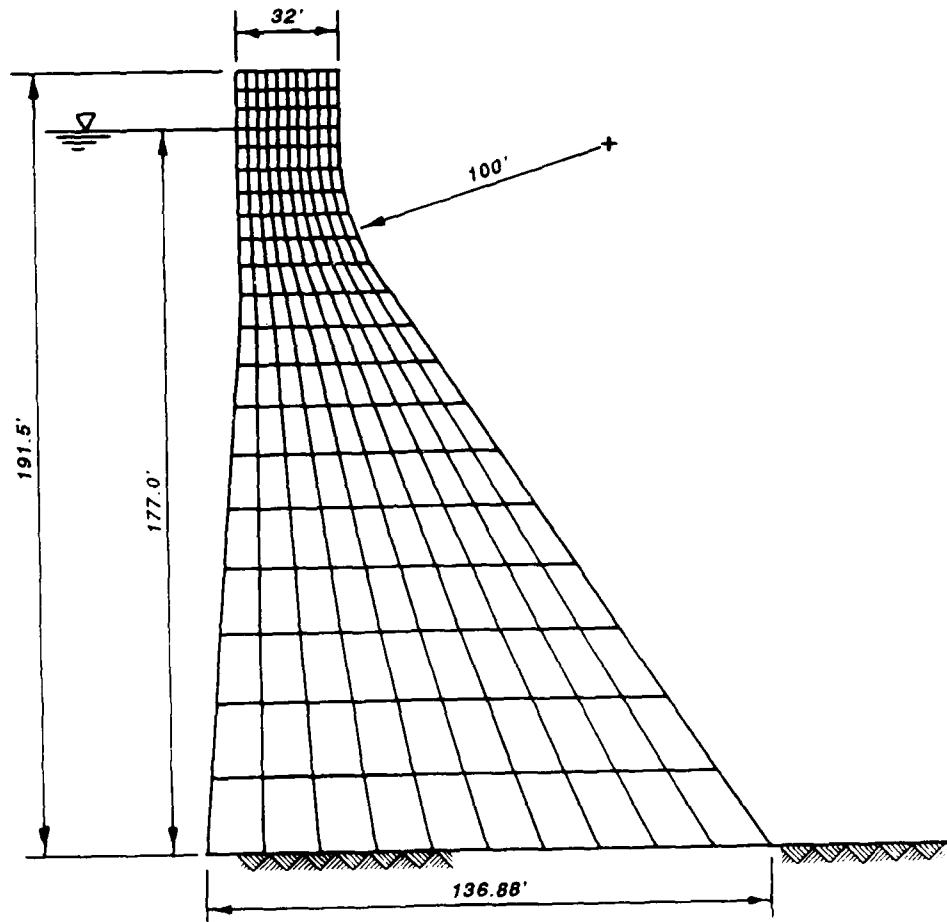
Figure F1. Grid of shortest nonoverflow monolith
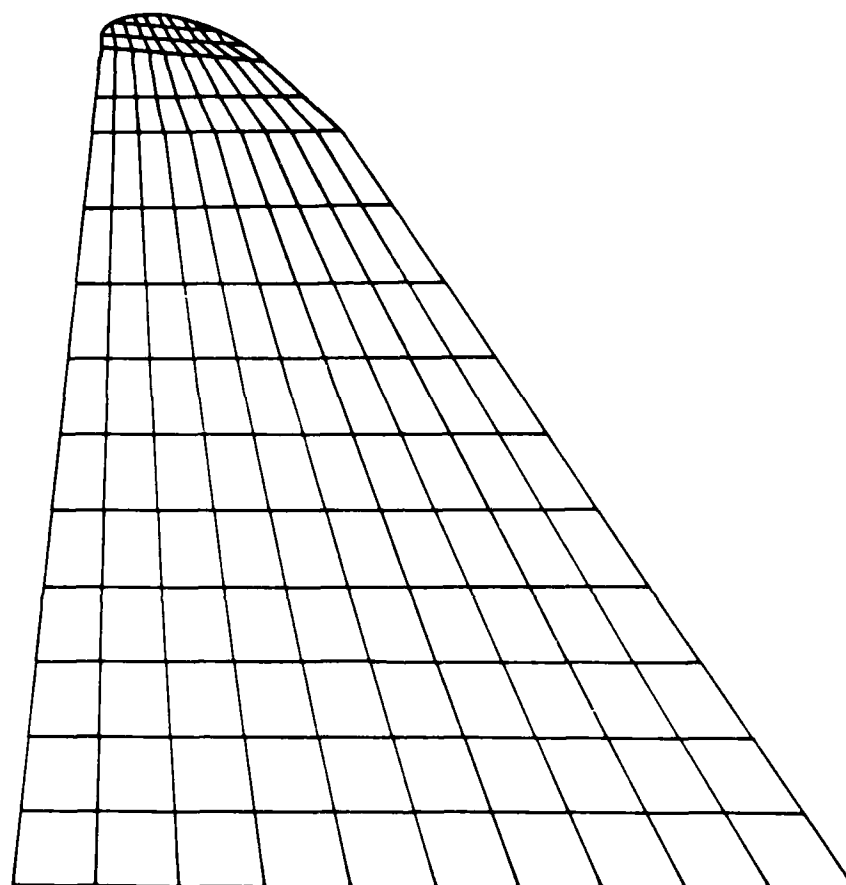
Figure F2. Grid of typical spillway sections

12'x 7' PIPE GALLERY

INSPECTION GALLERY

5'x 9' CONDUIT

INSPECTION GALLERY
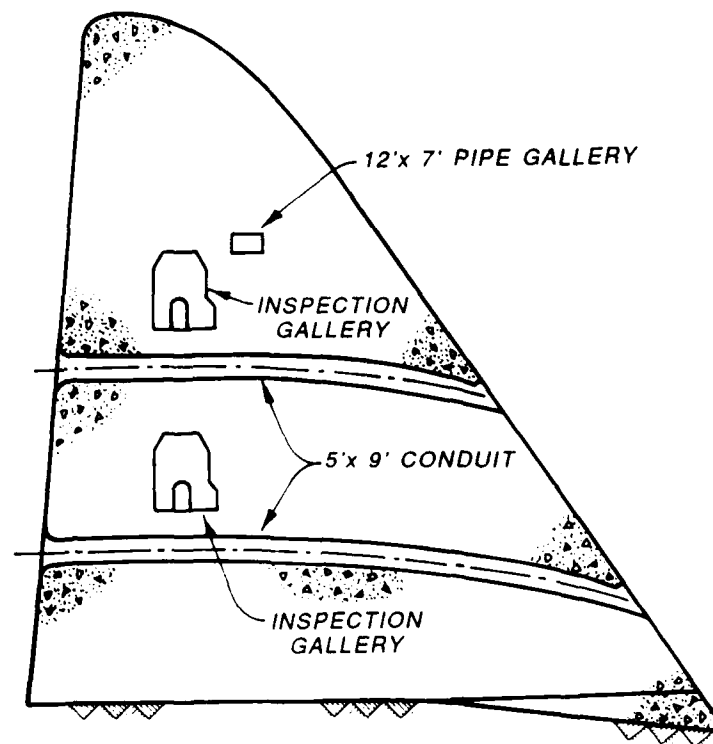
Figure 53. Typical spillway section
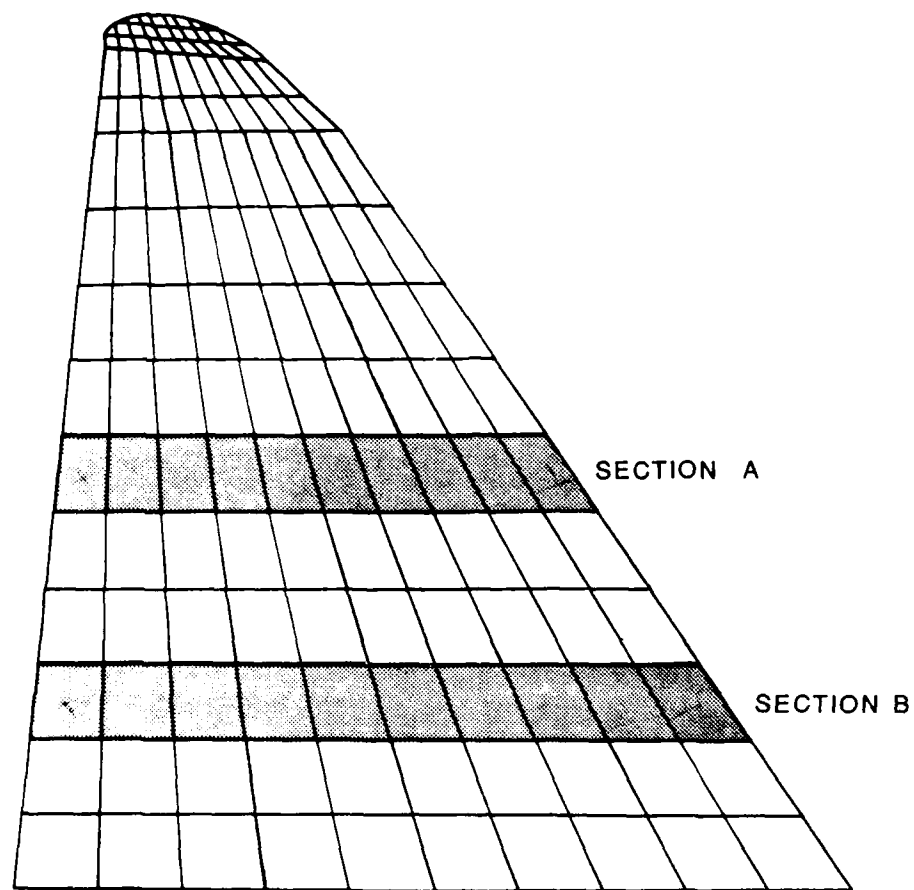
Figure 54. Spillway Sections A and B

Figure 5. Spillway section with pier

Figure F6. Pine Flat tower model

MATERIAL ID. NO.

SCALE

0   10   20 FT

EL 970'

PSEUDO - CONCRETE

SOLID CONCRETE
FOR PINE FLAT DAM

EL 870'
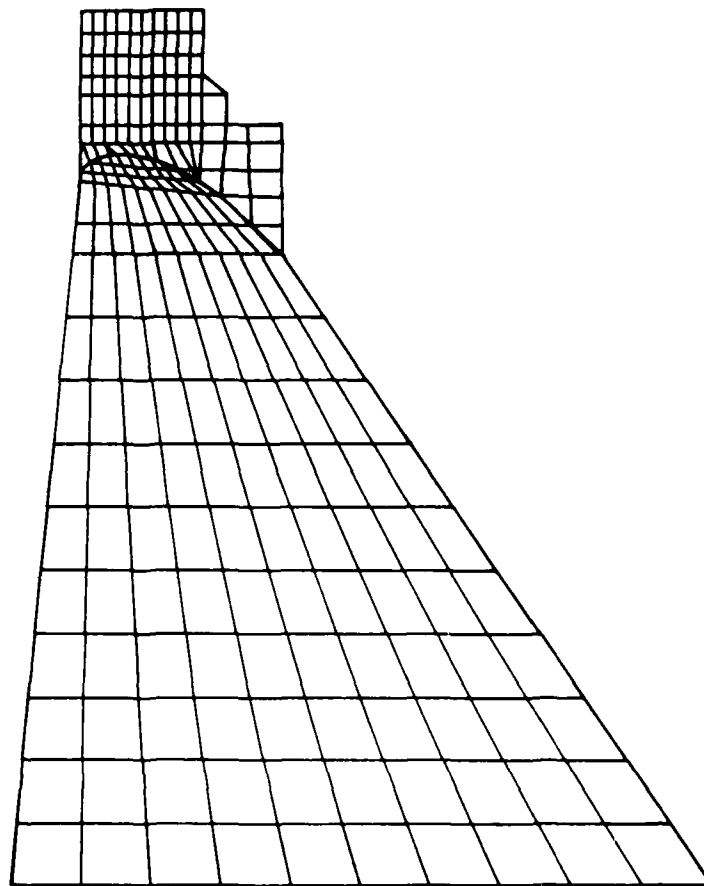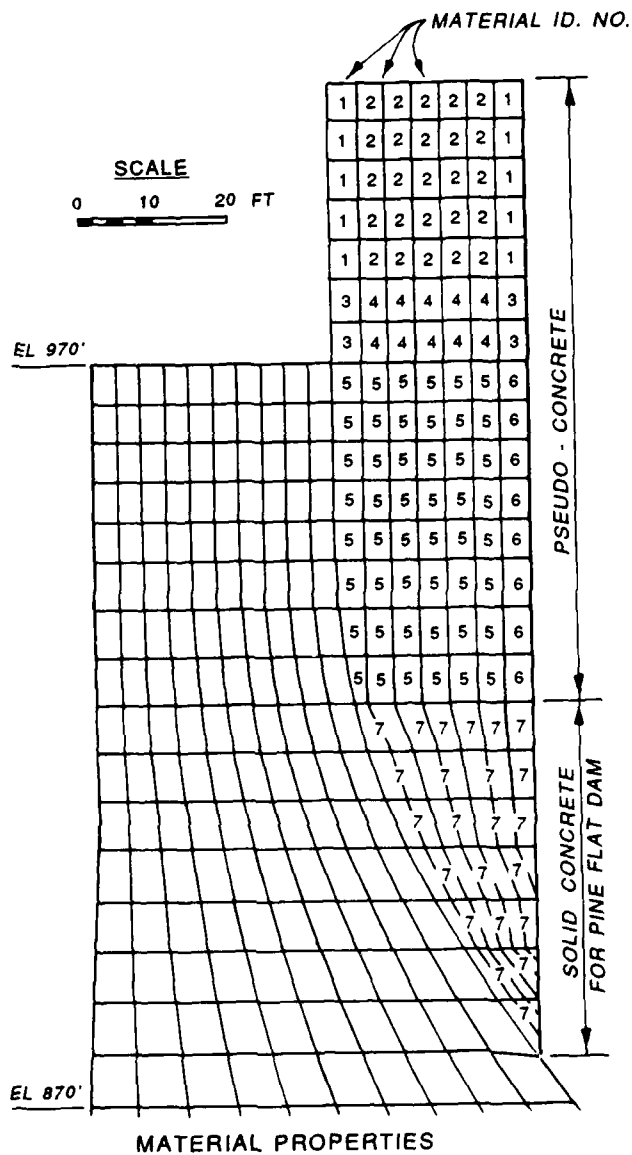
MATERIAL PROPERTIES

MATERIAL NO	M $(K\text{-}SEC^2/FT^4)$	E K/FT^2
1	0.001178	125608.6
2	0.0004193	44775.76
3	0.001509	173786.8
4	0.0005565	64907.85
5	0.0005621	65570.48
6	0.0015183	175652.3
7	0.004907	849600
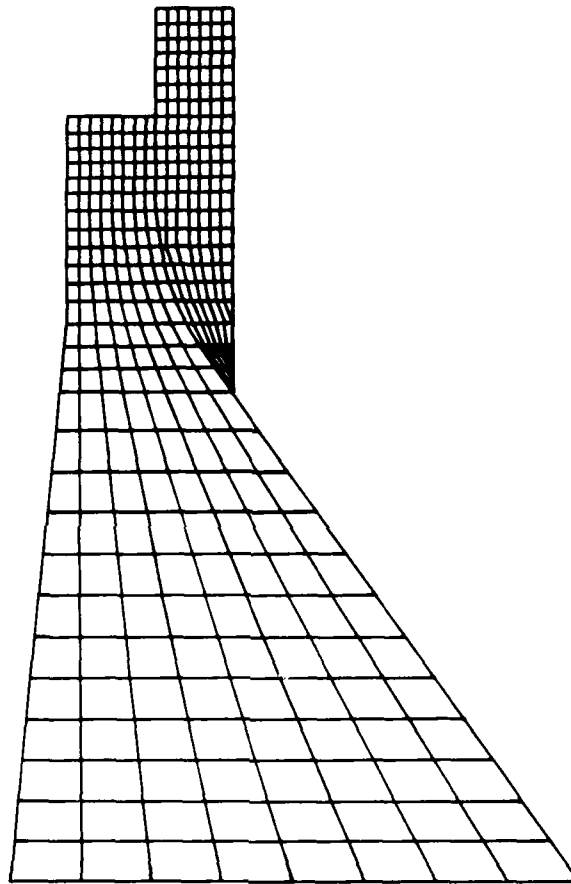
Figure F7. Tallest monolith with tower

4.6 HZ - WITHOUT TOWER

4.5 HZ - WITH TOWER
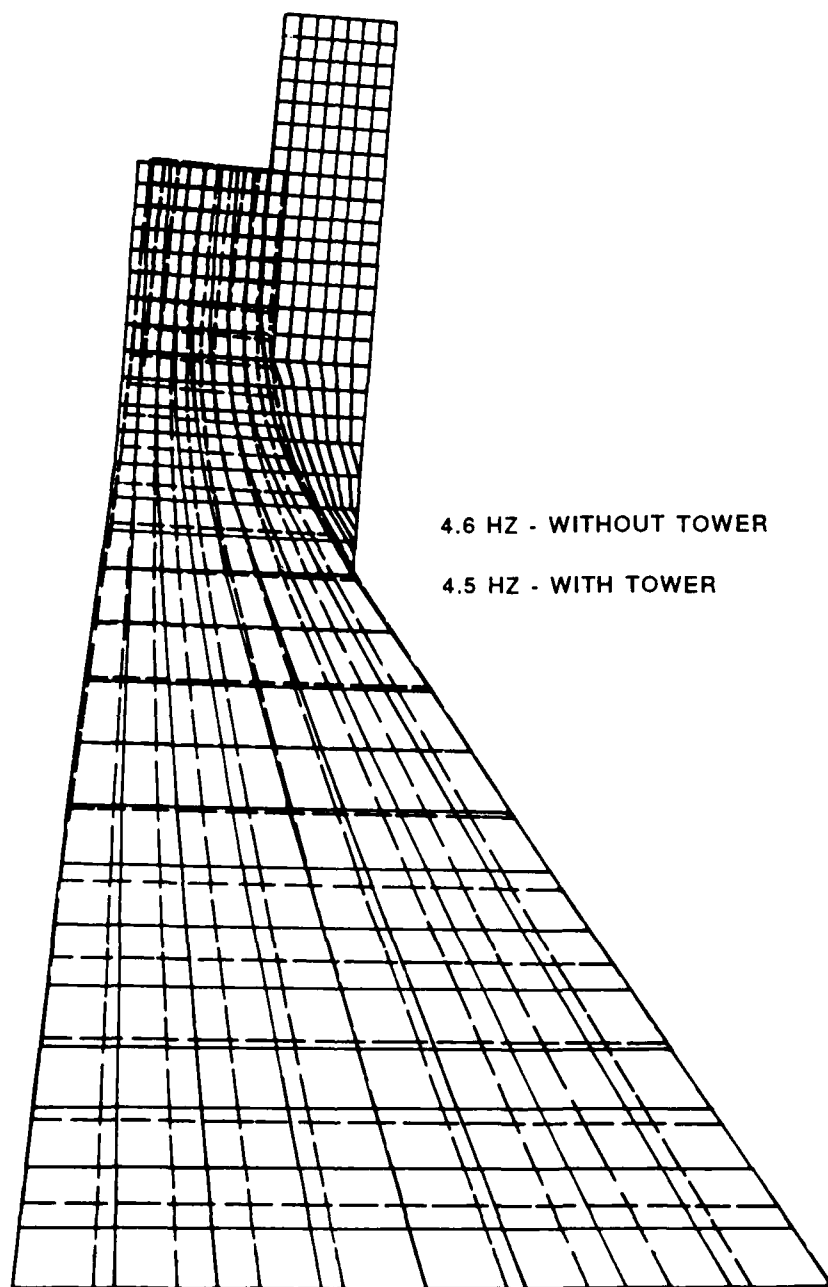
Figure F8.   Mode shape Number 1

9.5 HZ - WITHOUT TOWER

9.8 HZ - WITH TOWER
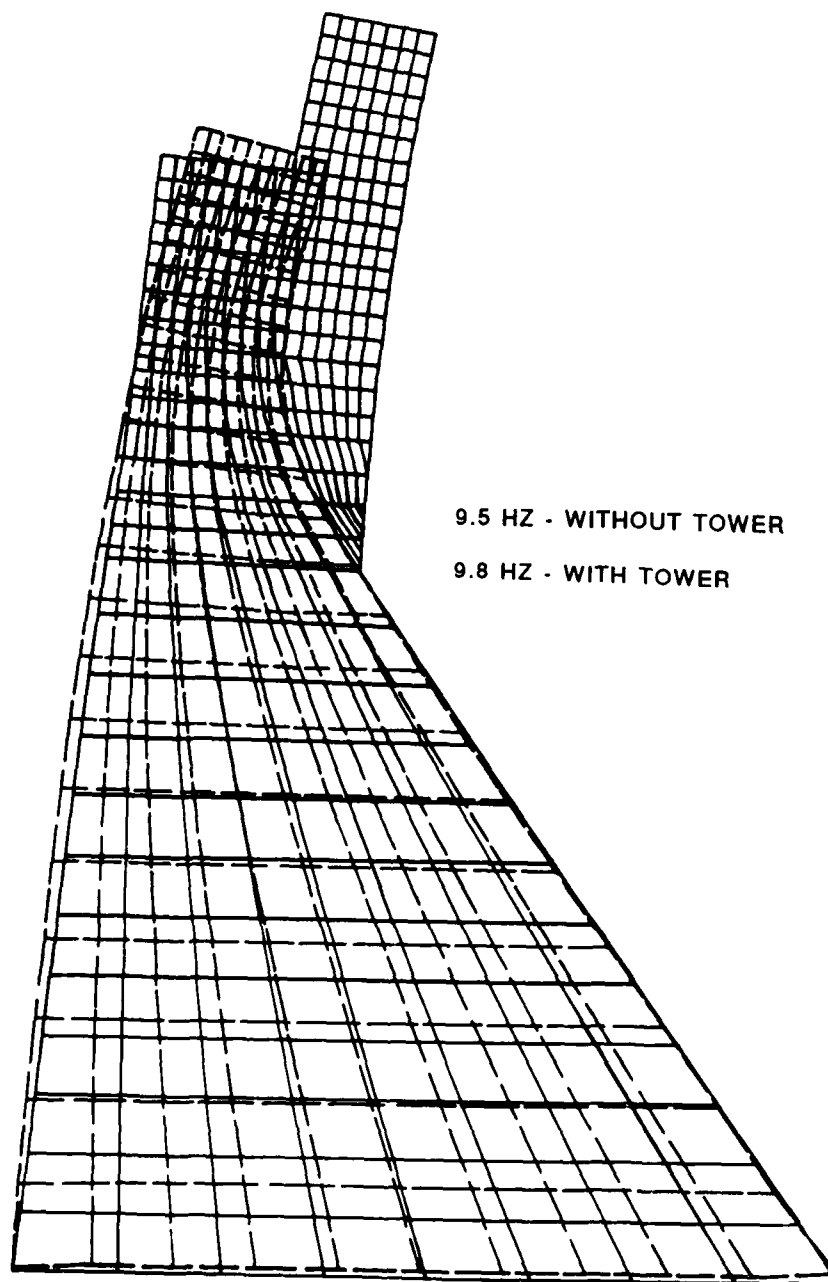
Figure F9. Mode shape Number 2

9.5 HZ - WITHOUT TOWER

10.0 HZ - WITH TOWER

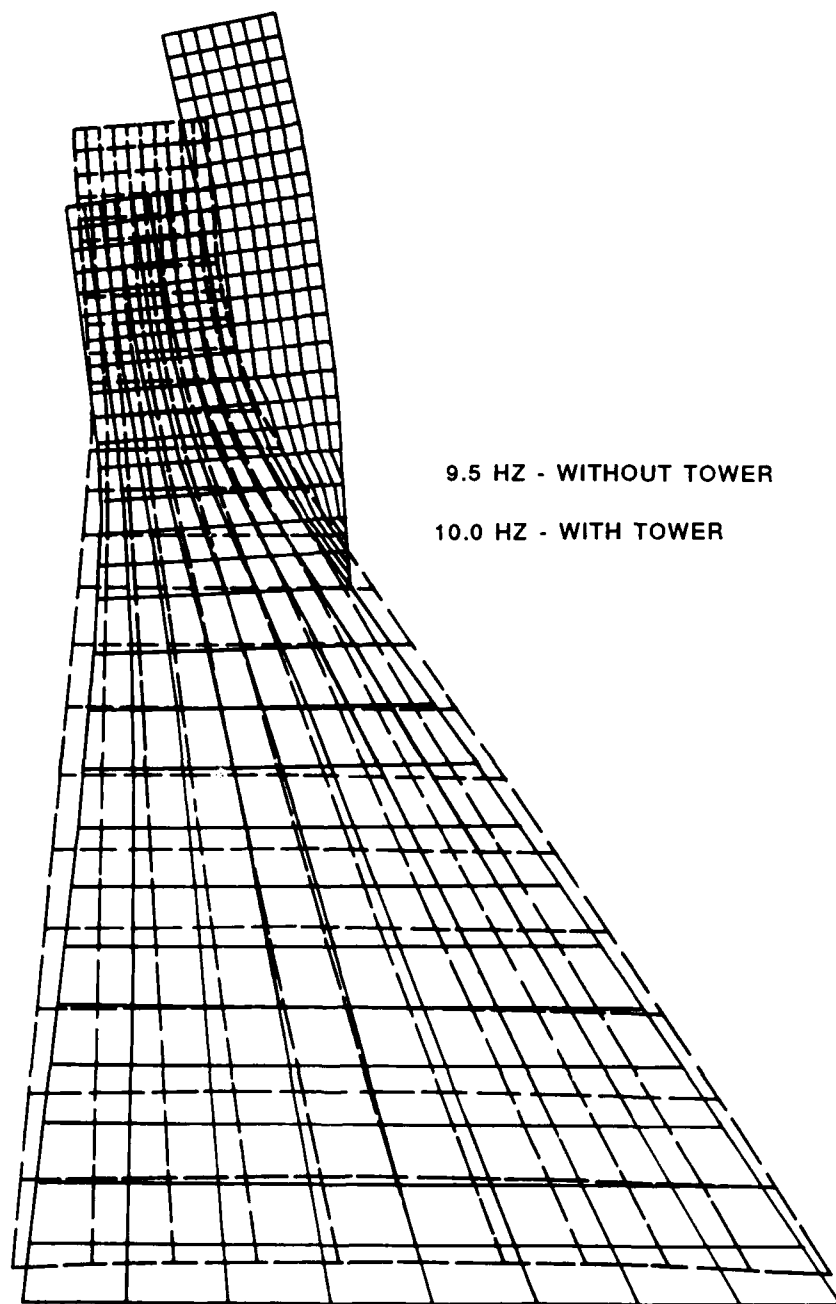Figure F10.   Mode shape Number 3

16.7 HZ - WITHOUT TOWER
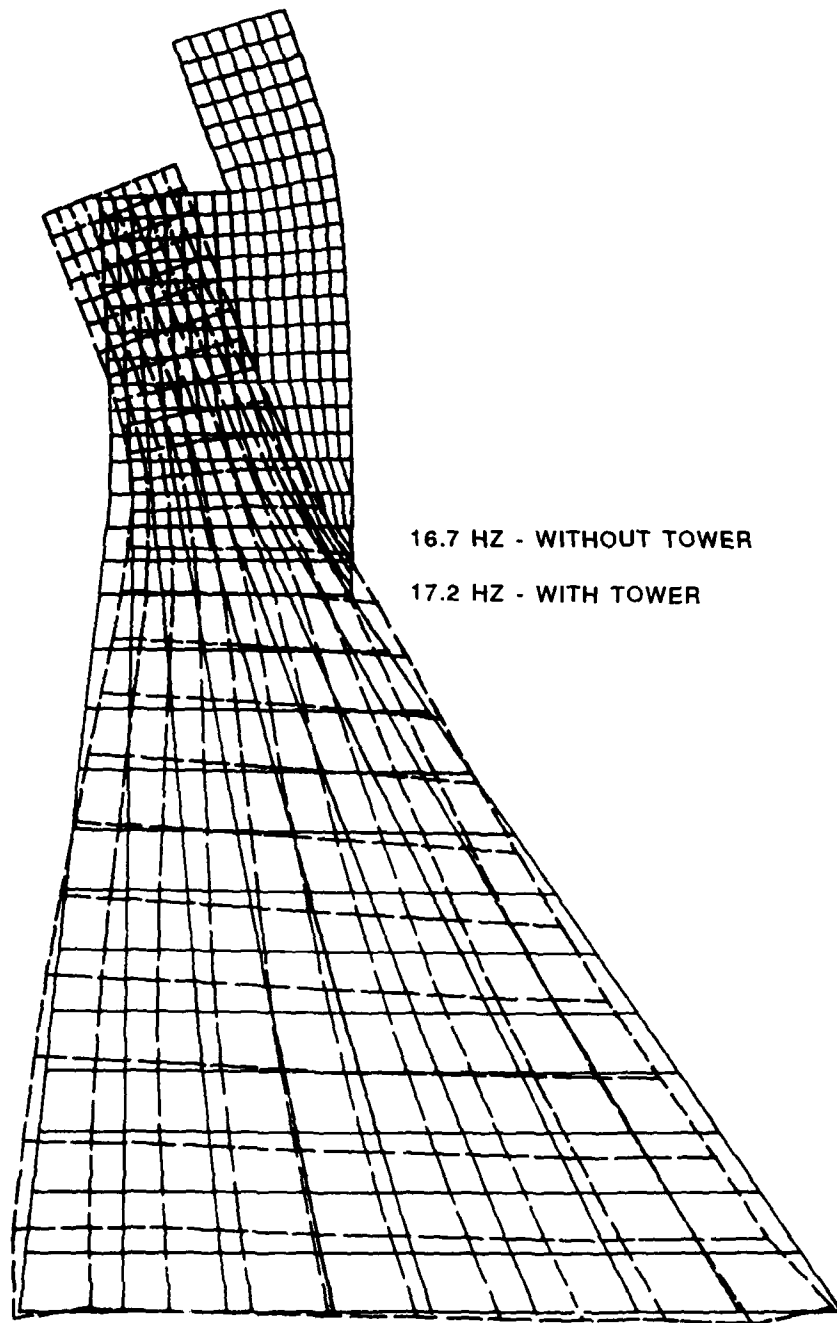
17.2 HZ - WITH TOWER

Figure F11. Mode shape Number 4

WITH AND WITHOUT
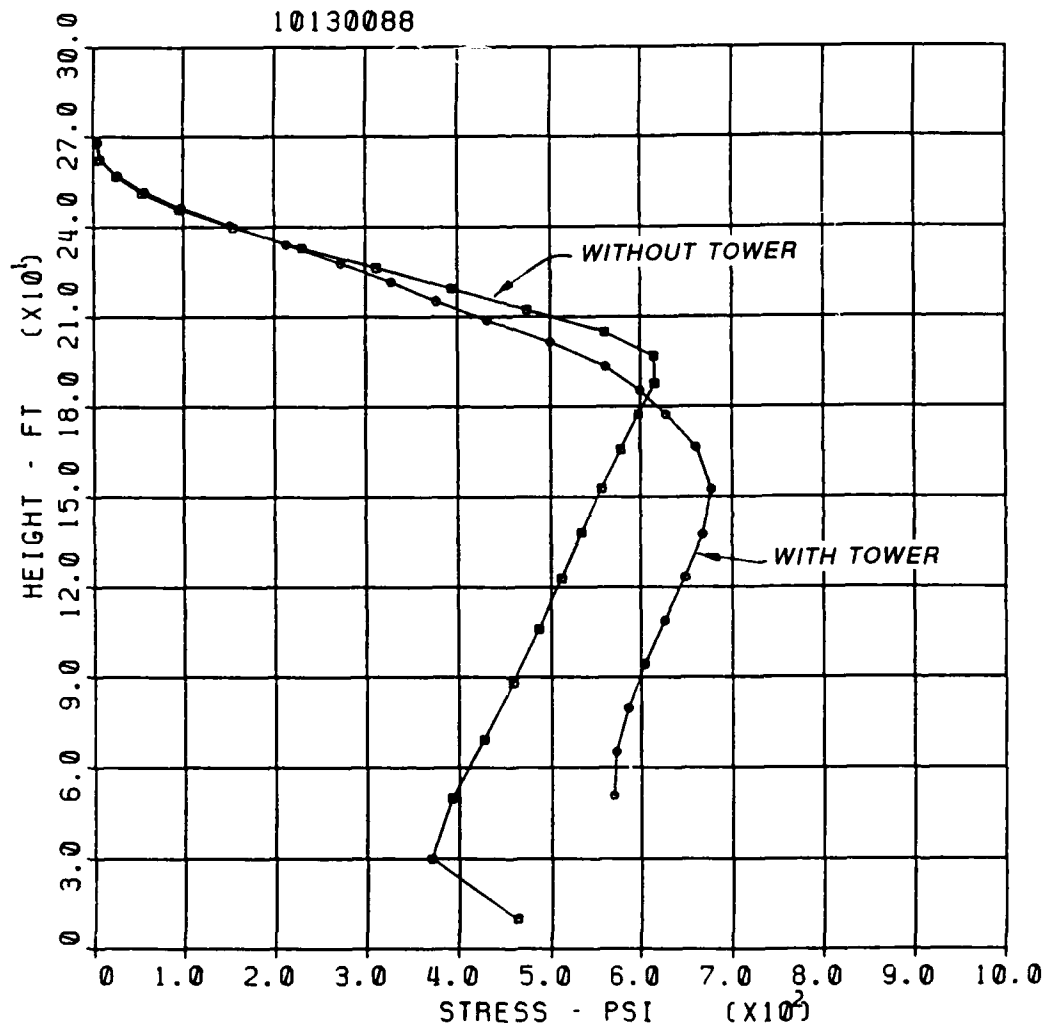# TOWER - SIGMA-1 UPST

Figure F12. Maximum principal stress along upstream face
with and without tower

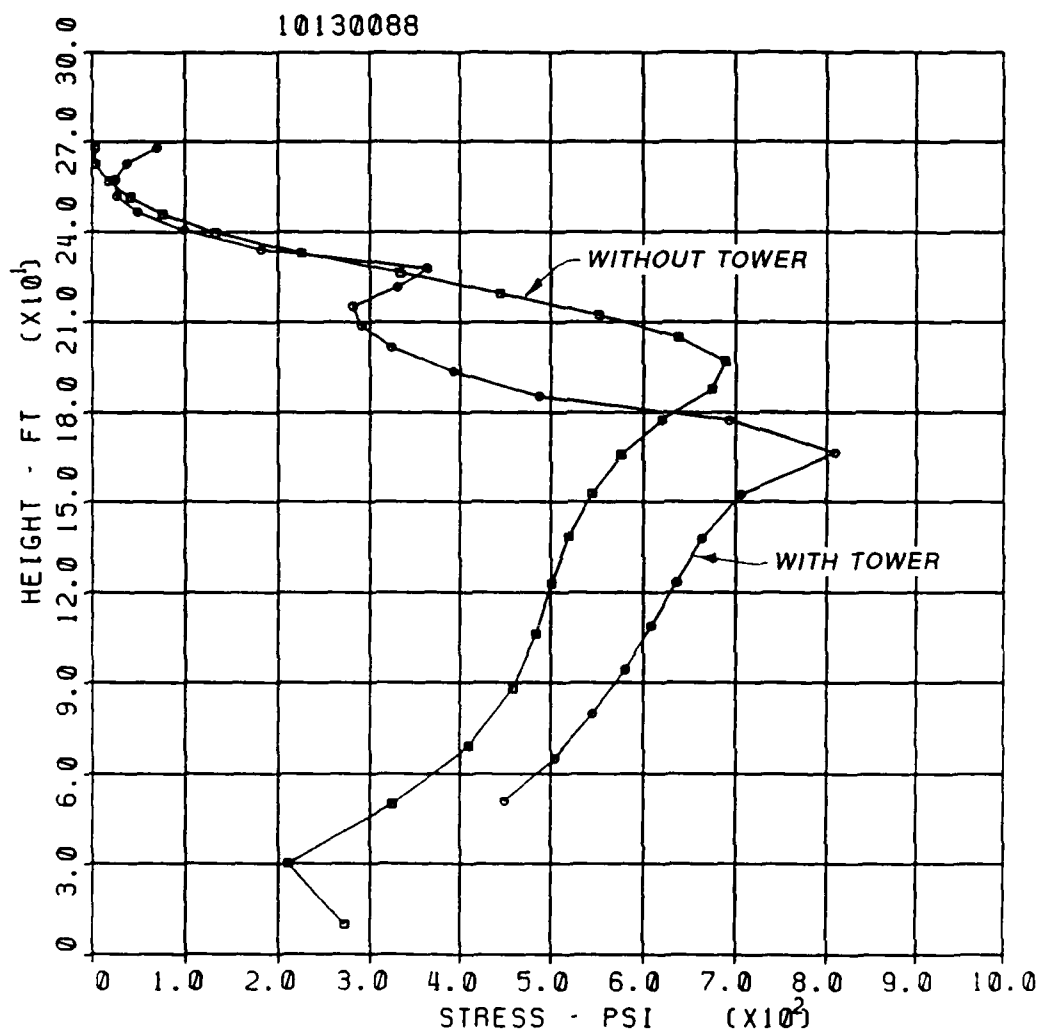# WITH AND WITHOUT
TOWER - SIGMA-1 DOWN

10130088

Figure F13. Maximum principal stress along downstream face
with and without tower
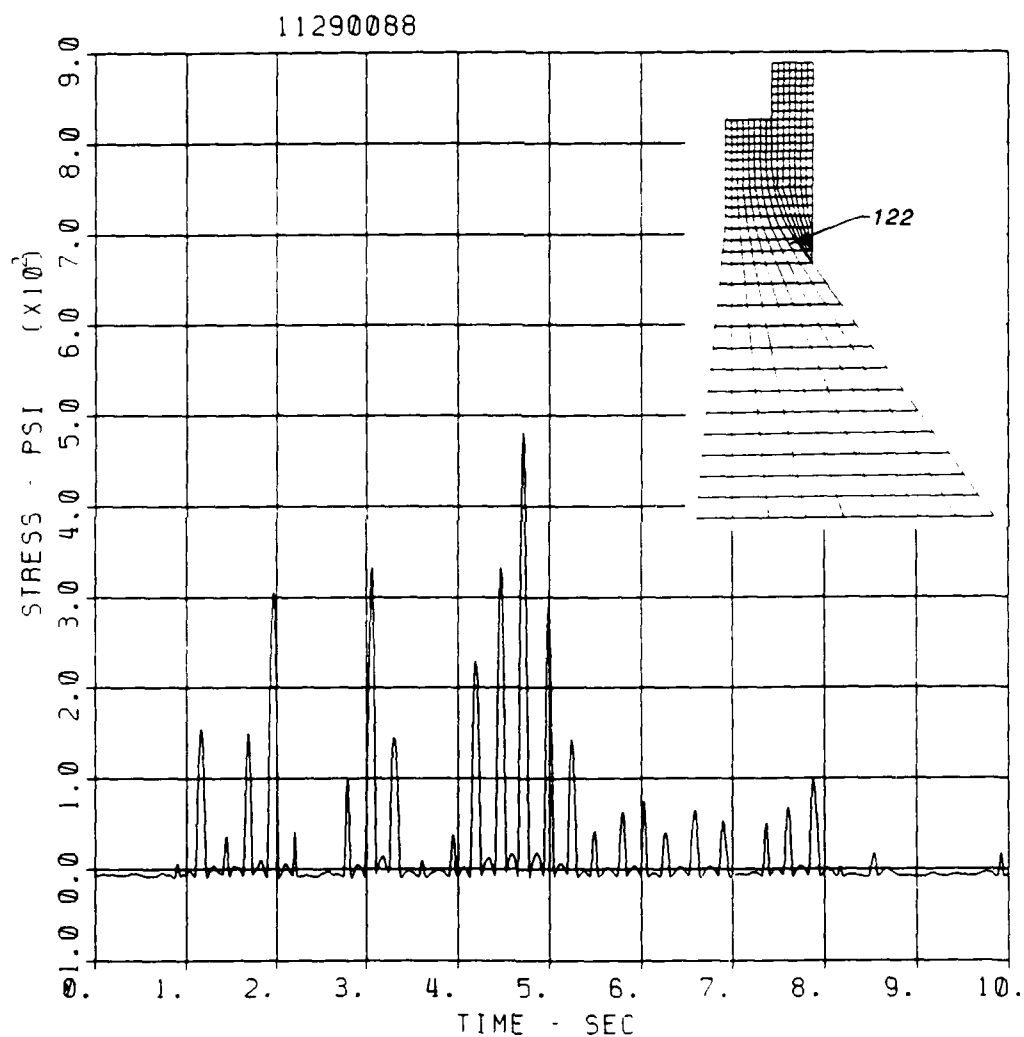
NONFLOW SECTION WITH SIGMA-1 ELE NO. 122

Figure F14. Principal stress time history for element 122

NONFLOW SECTION WITH
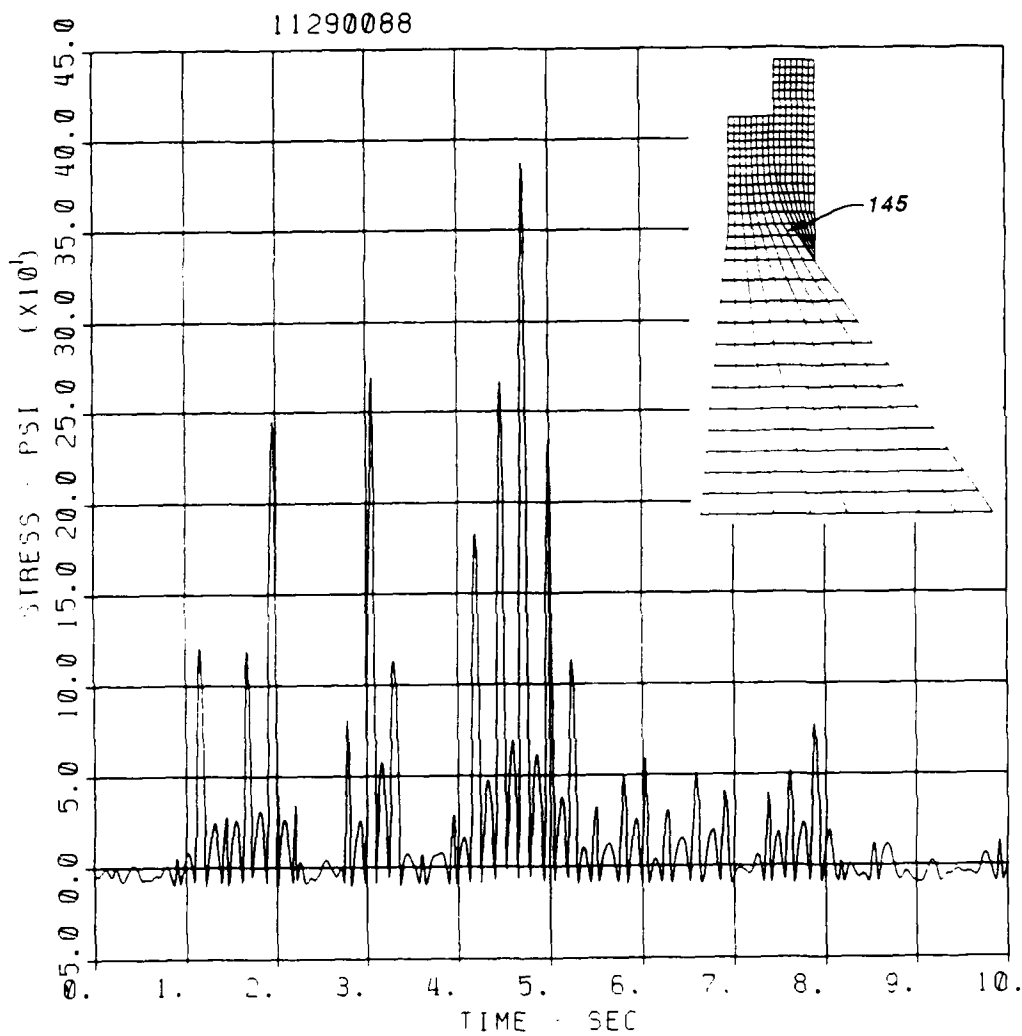# SIGMA-1 ELE NO. 145

Figure F15. Principal stress time history for element 145
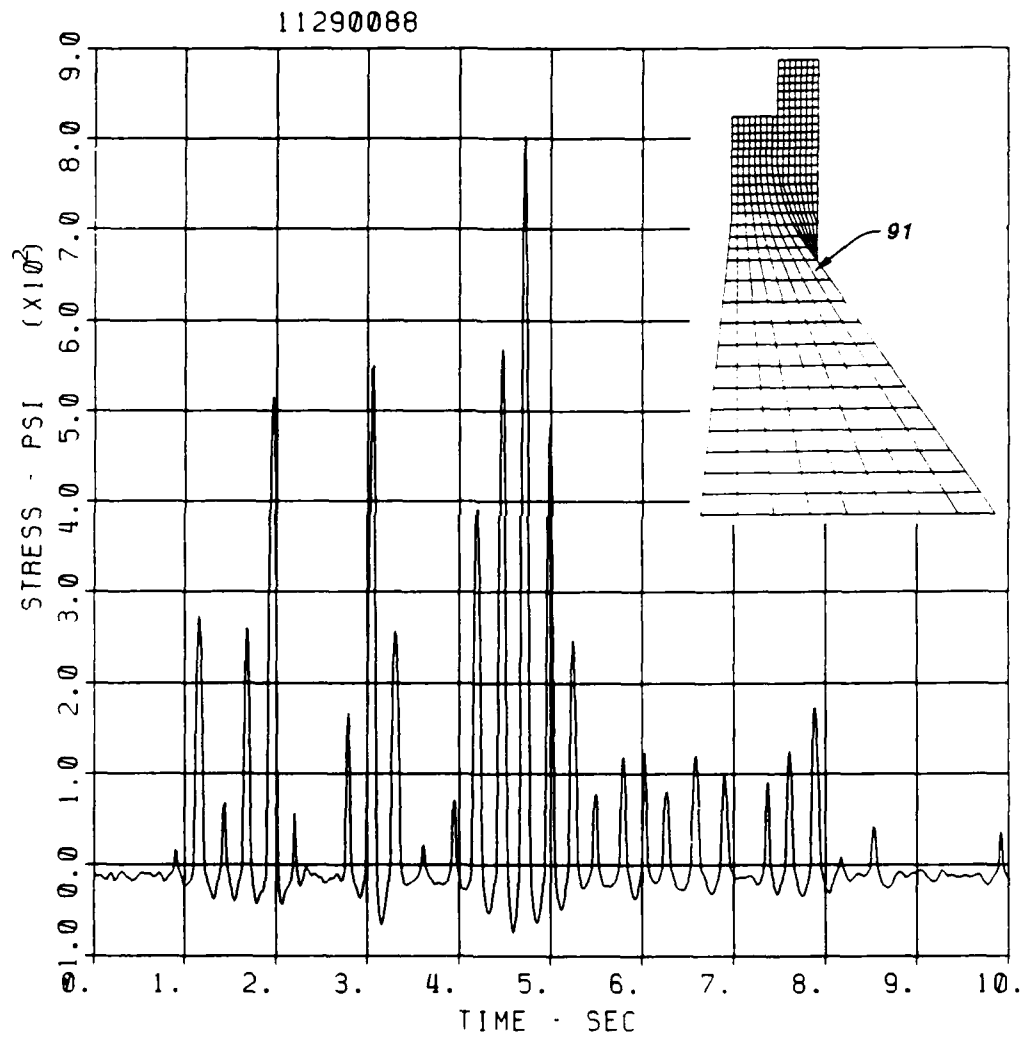
NONFLOW SECTION WITH
SIGMA-1 ELE NO. 91

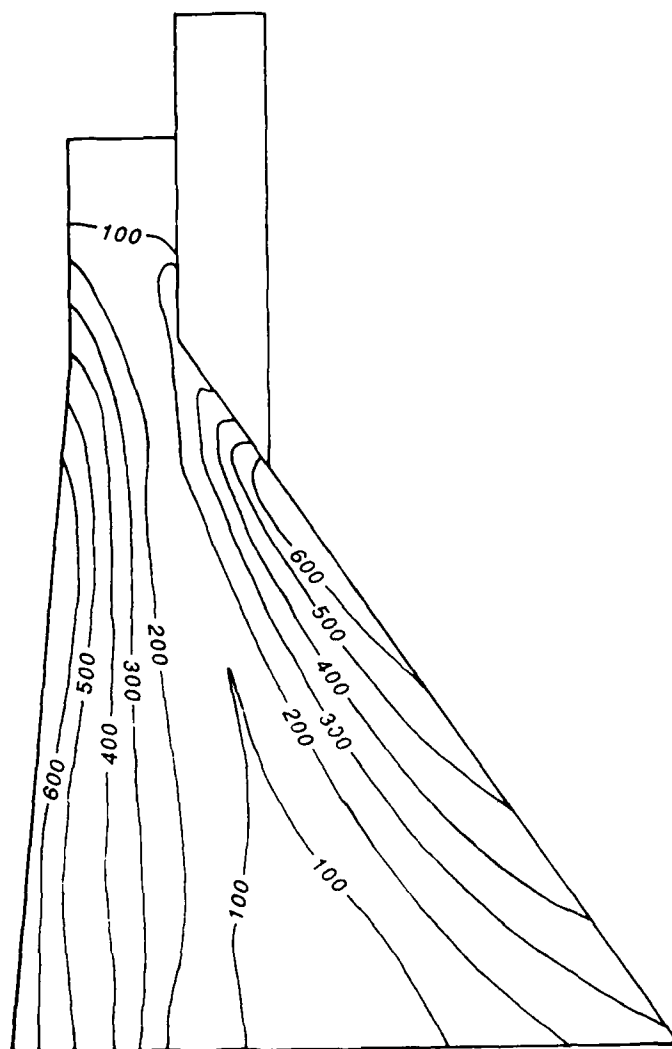Figure F16. Principal stress time history for element 91

Figure #17. Envelop of maximum principal stresses

APPENDIX G

STABILITY ANALYSIS

APPENDIX G: STABILITY ANALYSIS

1. A stability analysis was performed in accordance with ETL 1110-2-256 (US Army Corps of Engineers 1981). The analysis is limited to horizontal planes at or above the monolith-foundation plane. A seismic coefficient of 0.15g was obtained from ER 1110-2-1806 (US Army Corps of Engineers 1983) for computation of the inertia force due to acceleration of the monolith. The inertia force due to the reservoir water was determined using Westergard's formulation as described in ETL 1110-2-256. The uplift force at the foundation contact plane was based on the assumption of a straight-line variation from full headwater pressure at the heel to zero pressure at the toe for the reservoir level at the spillway flood pool elevation of 475.4. For planes within the monolith, the uplift force was based on a straight-line variation from 50 percent of full headwater pressure at the heel to zero pressure at the toe. A value of 158 lb/ft^3 was used for the density of the monolith concrete.

2. Uplift measurements are not available for the deeper monoliths. Based on an on-site inspection conducted in May 1988, the above assumptions for the uplift force are conservative. A grout curtain is in place to reduce the uplift force caused by headwater pressure. Very little water was entering the gallery area through the vertical foundation drainage holes during the on-site inspection, indicating that the grout curtain is effective. Most of the water entering the gallery was entering between vertical monolith joints. A chemical reaction between the concrete and rock caused blockage of many of the drainage holes. The holes were recently reopened, and there is practically no water entering the gallery through the holes.

3. Conservative values were used for the internal angle of friction (ϕ) and the cohesion intercept (c) for the granite foundation and the concrete. Based on values presented by Stagg and Zienkiewicz (1967), a ϕ-value of 51 degrees and a c-value of 1,400 psi were assumed for the granite. Smee (1967) presents results of triaxial compression tests on concrete conducted by other investigators. The data indicate that a value of ϕ = 32 degrees and a value of c = 1,500 psi correspond to a concrete compressive strength of 5,200 psi. Similarly, the data indicate that values of 35 degrees and 1,800 psi correspond to ϕ and c, respectively, for a concrete compression strength of 3,570 psi. Based on this data, a ϕ-value of 30 degrees and a c-value of 1,000 psi were taken to be conservative assumptions. These values resulted in

the computation of a lower shear strength for the concrete than for the granite. Therefore, the concrete was the controlling material at the foundation contact plane.

4. The resultant of the applied loads was found to lie outside the middle one-third of the base area, but within the base width. This is acceptable for instantaneous load cases such as due to seismic forces. However, the cohesive component of the sliding resistance should only include the portion of the base area that is in contact with the foundation material. Figure G1 is a sketch of the monolith section showing the forces involved in the stability analysis. The computations for determining those forces and the factor of safety against sliding are also summarized in Figure G1.

5. The analysis resulted in a factor of safety against sliding of approximately 2.4 at the foundation contact plane. The minimum required factor of safety for seismic loading conditions is 1.3 in accordance with ETL 1110-2-256. Table G1 gives values of the factor of safety for selected values of ϕ and c as an indication of the sensitivity of the factor of safety to those parameters. The factor of safety is much more sensitive to the c-value than to the ϕ-value. However, even the use of a c-value of 500 psi with a ϕ-value of 30 degrees results in a factor of safety of approximately 1.4 at the monolith-foundation contact plane, which is greater than the minimum allowable value of 1.3. Primarily, due to geometry and the reduction in uplift forces, the factor of safety increased to a value of greater than 14 at horizontal planes within the monolith.

Table G1

Sensitivity of F.S to ϕ and c

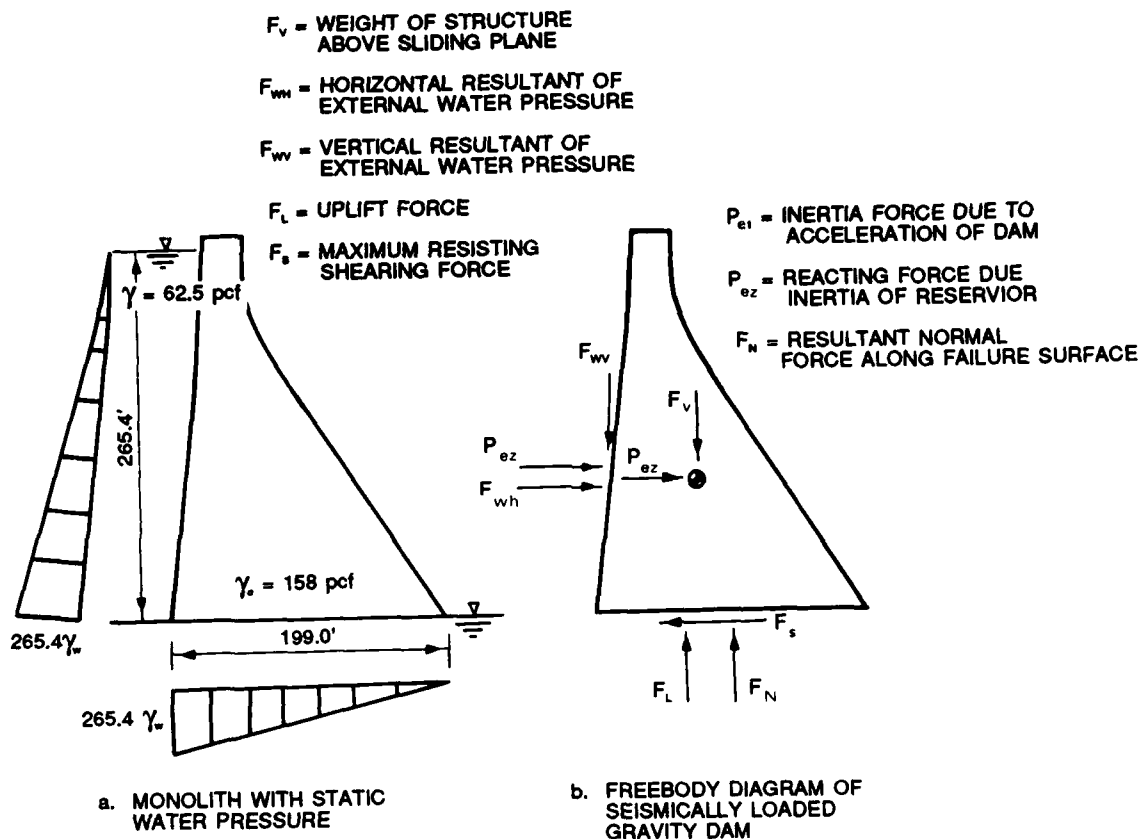| ϕ (degrees)	c (psi)	F.S
25	1000	2.30
28	1000	2.35
30	1000	2.40
30	500	1.44
30	1500	3.35

Appendix G References

Smee, D. J. 1967 (Oct). "The Effect of Aggregate Size and Concrete Strength on the Failure of Concrete Under Triaxial Compression," Civil Engineering Transactions.

Stagg, K. G. 1968. Zienkiewicz, O. C., Rock Mechanics Engineering Practice, John Wiley and Sons, New York.

US Army Corps of Engineers. 1981 (Jun). "Sliding Stability for Concrete Structures," Engineering Technical Letter (ETL) 1110-2-256, Washington, D.C.

US Army Corps of Engineers. 1983 (May). "Earthquake Design and Analysis for Corps of Engineers Projects," Engineer Regulation (ER) 1110-2-1806, Washington, D.C.

F_v = WEIGHT OF STRUCTURE ABOVE SLIDING PLANE

F_wH = HORIZONTAL RESULTANT OF EXTERNAL WATER PRESSURE

F_wv = VERTICAL RESULTANT OF EXTERNAL WATER PRESSURE

F_L = UPLIFT FORCE

F_s = MAXIMUM RESISTING SHEARING FORCE

P_e1 = INERTIA FORCE DUE TO ACCELERATION OF DAM

P_e2 = REACTING FORCE DUE INERTIA OF RESERVIOR

F_N = RESULTANT NORMAL FORCE ALONG FAILURE SURFACE

$\gamma = 62.5$ pcf

265.4'

$\gamma_e = 158$ pcf

265.4 γ_w

199.0'

265.4 γ_w

F_wv

F_v

P_ez

P_ez

F_wh

F_s

F_L

F_N

a. MONOLITH WITH STATIC WATER PRESSURE

b. FREEBODY DIAGRAM OF SEISMICALLY LOADED GRAVITY DAM

F_{WH} with $\gamma_w = 62.5$ pcf $= 2,201,151$ psf

F_{wv} with $\gamma_w = 62.5$ pcf $= 205,919$ psf

$F_L = \frac{1}{2}(199)(265.4)\gamma_w = 1,650,456$ psf

F_v with $\gamma_c = 158$ pcf $= 4,169,411$ psf

EARTHQUAKE FORCES (ETL 1110-2-256 / EM 1110-2-2200):

$P_{e1} = Ma = \frac{F_v}{g} \alpha g = \alpha F_v = 625,412$ lb/ft where $\alpha = 0.15$ (ER 1110-2-1806)

$P_{e2} = \frac{2}{3} C_e \alpha \gamma \sqrt{hy} = 372,613$ lb/ft where $C_e = \dfrac{51}{\sqrt{1 - 0.72\left(\frac{h}{1000\, t_e}\right)^2}} = 52.40$

$h = 270.5'$
$t_e = 1$ sec
$y = 265.4'$

Resultant of all forces in dam $= 4,202,340$ lb/ft and passes through base at 184.7' from heel

Since the cohesive component of the sliding resistance should include only the portion of base in contact with foundation (42.3'), the factor of safety is:

$F.S. = \dfrac{F_N \tan\varphi + 42.3'\, c}{\Sigma \text{Horizontal Forces}} = \dfrac{F_s}{\Sigma \text{Hor. Forces}}$ where $\varphi = 30°$, $c = 10,000 = 144,000$ psf

$F.S. = \dfrac{(2,724,874\, psf)\tan 30° + 42.3'(144,000\, lb/ft^2)}{3,177,170\, lb/ft^2} = 2.4$

Figure G1. Sliding stability